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Quantifying Annual Bridge Cost by Overweight Trucks in South Carolina

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QUANTIFYING ANNUAL BRIDGE COST BY OVERWEIGHT TRUCKS IN SOUTH CAROLINA

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ABSTRACT

With the economic development in recent decades, more trucks including overweight trucks are operating on the highways. As a result, many bridges are expected to carry more loads than they did in previous years. The impact of overweight trucks on existing bridges has been an urgent concern for the South Carolina Department of Transportation (SCDOT). There is a pressing need to quantify the annual bridge cost in South Carolina caused by trucks and, in particular, overweight trucks so that the SCDOT and the state legislators can determine the appropriate fee structure for operating overweight trucks. This research focused on quantifying the annual bridge cost in South Carolina caused by trucks and especially the overweight trucks. The annual bridge cost quantified in this study included two parts: the damage cost and the maintenance cost. Since the bridge damage cost is mainly attributed to repeated loading caused by the truck traffic, the fatigue analysis was utilized to quantify the bridge damage cost.

Four Archetype bridge models were developed and used as surrogate models to represent the 9,271 bridges in South Carolina. The weigh-in-motion data, size and weight inspection violations data and SCDOT overweight truck permit data were used to develop truck models. Archetype bridges with different truck models were analyzed using a finite element (FE) software called LS-DYNA. Using the stress ranges calculated from the FE analyses, annual bridge fatigue damage was estimated and the associated annual bridge damage cost in South Carolina was computed using the bridge replacement costs.

The total asset value or replacement cost for the South Carolina bridges is approximately $9.332 billion dollars (2011 US Dollar) and the annual bridge damage cost

(not including the maintenance cost) is estimated to be $29.35 million dollars (2011 US Dollar). Combined with the annual bridge maintenance cost, the total annual bridge cost in South Carolina is $35.795 million dollars (2011 US Dollar).

Based on the damage contribution and percentage of the overweight trucks in the overall truck population, the annual bridge cost allocated to the overweight trucks (including bridge damage costs and bridge maintenance cost) is found to be $8.484 million dollars.

To assist the SCDOT in establishing a new overweight permit fee structure, unit costs (cost per mile) were computed using the VMT (vehicle miles traveled) of individual truck models of different axle configurations and gross weights. It has been observed that the relationship between unit cost and gross vehicle weight is highly nonlinear.

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CHAPTER ONE

INTRODUCTION

According to the 2009 American Society of Civil Engineers (ASCE) Infrastructure Report Card, more than 26% bridges were determined to be either "functionally obsolete" or "structurally deficient" nationwide (ASCE 2009). For those structurally deficient bridges in which their structural capacities have been severely weakened, the state departments of transportation have to post reduced weight limit on them. At the same time, according to a report by the Federal Highway Administration (FHWA), there was a 1.68 percent annual increase in the amount of vehicles from 1980 to 2004 and only 0.21 percent increase in new highway lane miles from 1980 to 2003 (FHWA 2007). This fast growing truck loading demand certainly exacerbated the deterioration of bridges.

Within all these truck loadings, the greatest concern of many state departments of transportation is the overweight truck loading which causes more bridge deterioration than other normal weight truck loadings. Researchers have conducted studies of bridge damage by overweight trucks in different states (Chotickai and Bowman 2006a; Altay et al. 2003). The same concerns exist in South Carolina, where there is a pressing need to quantify the annual bridge cost in South Carolina caused by trucks and, in particular, overweight trucks so that the South Carolina Department of Transportation (SCDOT) and the state legislators can determine a rational fee structure for operating overweight trucks.

This objective of this research is to quantify the annual bridge cost in South Carolina caused by trucks and especially overweight trucks. The annual bridge cost included two parts: the damage cost and the maintenance cost. Since the bridge damage cost is mainly attributed to repeated loading caused by the truck traffic, the bridge damage cost was estimated based on the fatigue loading.

In this research, the information of all bridges in South Carolina was obtained from the National Bridge Inventory database (NBI 2012), which is maintained by the Federal Highway Administration. For analysis purpose, bridges were grouped into archetypes based upon the construction material, structural system and span length. Since the predominant bridge types are reinforced concrete and prestressed concrete bridges, analyses were performed for these two types of bridges. In order to quantify the relative damages caused by different types of trucks, a series of representative truck models with different gross weights and axle configurations were developed. The gross truck weight distribution and axle spacing were determined using the South Carolina Department of Public Safety weigh-in-motion data (SCDPS 2012a), South Carolina Department of Public Safety size and weight inspection violations data (SCDPS 2012b) and the South Carolina Department of Transportation (SCDOT) overweight truck permit data (SCDOT 2012b). Bridge responses during the passage of normal weight trucks and overweight trucks were determined through advanced dynamic analysis using a finite element (FE) software called LS-DYNA (LS-DYNA 2010). Using the stress ranges calculated from the FE analyses, annual bridge fatigue damage was estimated and the associated annual bridge damage cost in South Carolina was computed using the bridge replacement costs. Once the annual bridge fatigue damage cost was obtained, the annual bridge maintenance cost (SCDOT 2012c) was added to the damage cost to obtain the total annual bridge cost in South Carolina. Finally, the annual bridge cost allocated to the overweight trucks was calculated based on both the damage contribution of overweight trucks and percentage of overweight trucks in the overall truck population. To assist the SCDOT in establishing a new overweight permit fee structure, unit costs (cost per mile) were computed using the vehicle miles traveled (VMT) of individual truck types of different axle configurations and gross vehicle weights.

Research Objectives

Here are the major objectives of this research:

1 Quantify the annual bridge cost in South Carolina

  1.1 Cost by all trucks

  1.2 Cost allocated to overweight trucks

2 Quantify unit costs (cost per mile)

  2.1 Overweight trucks unit costs

  2.2 Super-load trucks unit costs

Thesis Organization

The following chapters of this thesis discuss the above topics in detail. Chapter 2 presents the literature review. Chapter 3 presents the research methodology. Chapters 4 and 5 discuss the development of truck models and Archetype bridges, respectively. Chapter 6 shows the details of the FE models for Archetype bridges and presents the analysis results. Chapter 7 discusses the determination of the total bridge asset value as well as the replacement cost for individual bridges in South Carolina. Chapter 8 discusses the determination of bridge fatigue life. Chapter 9, 10 and 11 explain the process of determining the annual bridge cost, overweight truck bridge cost and super-load truck bridge costs respectively. Lastly, conclusions and summaries are provided in Chapter 12.

CHAPTER TWO

LITERATURE REVIEW

Introduction

In recent years, the problem of bridge deterioration is gaining more and more attention for several reasons. The first reason is that in the past decade, the increases of population, traffic flow and car ownership were much faster than the development of road network. One recent study shows that there was a 1.06 percent annual increase in the population from 1980 to 2004, a 3.11 percent annual increase in the gross domestic product (GDP) and a 1.68 percent annual increase in the amount of vehicles, while there was only a 0.21 percent increase in new highway lane miles over the same period (FHWA 2007). The increase of traffic, particularly the freight traffic, was much faster than the growth of bridge network. The increased traffic frequency means our bridges may suffer more damage than in the past.

Second, with the economic development in recent decades, more trucks with increased loads are operating on highways. As a consequence, bridges are expected to carry more loads than they did in the past. In order not to deter economic growth, many states are allowing more overweight trucks to operate on their highway routes. This fast growing truck loading raises concern over the additional bridge damage cost caused by overweight trucks.

Lastly, the high costs associated with highway and bridge maintenance combined with recent economy downturn raises concerns regarding the large stock of aging bridge infrastructure in the United States. According to the 2009 ASCE Infrastructure Report

Card, more than 26% of the bridges were deemed either “functionally obsolete” or “structurally deficient” (ASCE 2009). Based on the estimation of the American Association of State Highway and Transportation Officials (AASHTO) in 2008, $140 billion dollars are needed to repair all the deficient bridges in the country (ASCE 2009). In order to keep the current bridge conditions, an annual investment of $13 billion and a total investment of $650 billion in 50 years are needed (ASCE 2009).

The issues discussed are faced by many states including South Carolina. Considering the above reasons, there is a pressing need to quantify the annual bridge cost in South Carolina caused by trucks and, in particular, overweight trucks so that the SCDOT and the state legislators can determine the appropriate fee structure for operating overweight trucks. In this study, the annual bridge cost was grouped into two components: the damage cost and the maintenance cost. Since the bridge damage cost is mainly attributed to repeated loading caused by the truck traffic, the fatigue analysis was utilized to quantify the bridge damage cost.

As stated previously, reinforced concrete and prestressed concrete bridges are the predominant bridge types in South Carolina. In the following sections, the fatigue behavior of reinforcement concrete and prestressed concrete bridges and fatigue design specifications in AASHTO Load and Resistance Factor Design (LRFD) Specification are discussed.

Reinforcement Concrete Bridge Fatigue

Rebar is a very important component in reinforcement concrete bridges. The fatigue behavior of rebars has been studied by others (e.g. Helgason et al. 1976). Through experimental investigations of 353 reinforced concrete beams, Helgason et al. (1976) concluded that factors including stress range, yielding stress, minimum stress, bar diameter, grade of bar and bar geometry affected the fatigue strength of rebars. Among these factors, the stress range was found to be the most critical factor in determining the rebar's fatigue strength and fatigue life (Helgason et al. 1976).

Minimum stress was found to be the second most important factor that affects the fatigue life. Helgason et al. (1976) found that, when the stress range of a rebar was above the endurance limit (i.e. it had a finite fatigue life), an increase in the minimum stress led to a decrease in the rebar fatigue strength when this minimum stress was tensile stress. On the other hand, an increase in the minimum stress led to an increase in the rebar fatigue strength when this minimum stress was compressive stress (Helgason et al. 1976).

When rebar had a finite fatigue life, the rebar nominal diameter was found to have a nonlinear effect on fatigue strength and grade of rebar was found to have a linear effect on rebar fatigue strength (Helgason et al. 1976).

Although it was determined that rebar geometry had a statistically significant effect on the rebar fatigue strength, rebar geometry was less important than the other factors mentioned above (Helgason et al. 1976).

Researchers also found the depth of beam, concrete strength, concrete elastic modulus, and beam dimensions had negligible effects on the rebar fatigue properties in straight reinforced concrete beam (Helgason et al. 1976).

Based on linear regression of the fatigue test results, Helgason et al. (1976) found that the fatigue life of Grade 60 rebars can be expressed in terms of the stress range:

$$\mathrm{Log}\,\mathrm{N} = 6.969 - 0.0383 \times \sigma \tag{2.1}$$

where

N: fatigue life in number of stress cycles

σ: rebar stress range in ksi

The above equation could explain around 76.8% variation of the entire test database and the standard deviation of this equation was 0.1657 (Helgason et al. 1976).

Alternatively, the fatigue life of rebars can be more accurately estimated with additional parameters including stress range, minimum stress, rebar yield stress and nominal bar diameter (Helgason et al. 1976):

$$\mathrm{Log}\,\mathrm{N} = 4.419 - 0.0392 \times \sigma - 0.013 \times \sigma_{min} + 0.0079 \times \mathrm{G} + 7.8059 \times \mathrm{D}_{nom} - 8.4155 \times {\mathrm{D}_{nom}}^2 + 2.799 \times {\mathrm{D}_{nom}}^3 \tag{2.2}$$

where

N: fatigue life in number of stress cycles

σ_{min}: minimum stress during stress cycle in ksi

G: rebar yield strength in ksi

D_{nom}: nominal rebar diameter in inches

This equation had a standard deviation of 0.1064 and it could explain around 90.7% variation of the entire test database (Helgason et al. 1976). Equation (2.2) was utilized in this study since it is more accurate than Equation (2.1).

Figure 2.1 shows a typical rebar fatigue curve, expressed in terms of the stress range (S) versus the number of cycles (N). The fatigue curve is commonly known as the S-N curve. According to Helgason et al. (1976), there is a limiting stress range (endurance limit), below which the rebar is assumed to have infinite fatigue life (Figure 2.1).

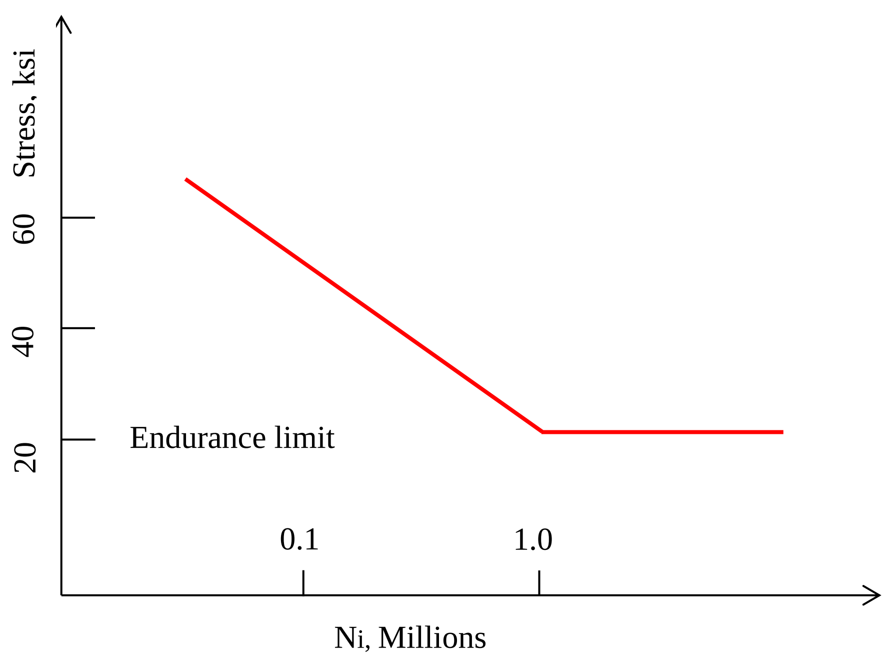


Figure 2.1: Rebar S-N Curve (Helgason et al. 1976).

From Figure 2.1 one can see that the endurance limit is around 20 ksi. A rebar is expected to be able to sustain unlimited number of cycles if its stress range is below this limit (Helgason et al. 1976). Note that the fatigue experiments by Helgason et al. (1976) were tested to a maximum of five million cycles. However, a recent fatigue study with large number of cycles (Giga-cycles) (Bathias and Paris 2005) shows that there is a further fatigue strength drop beyond the endurance limit determined by Helgason et al. (1976) (see Figure 2.2). The slope of the fatigue curve in the Giga-cycle region is similar to that of the High-cycle fatigue region. More details on the Giga-cycle fatigue can be found in Bathias and Paris (2005).

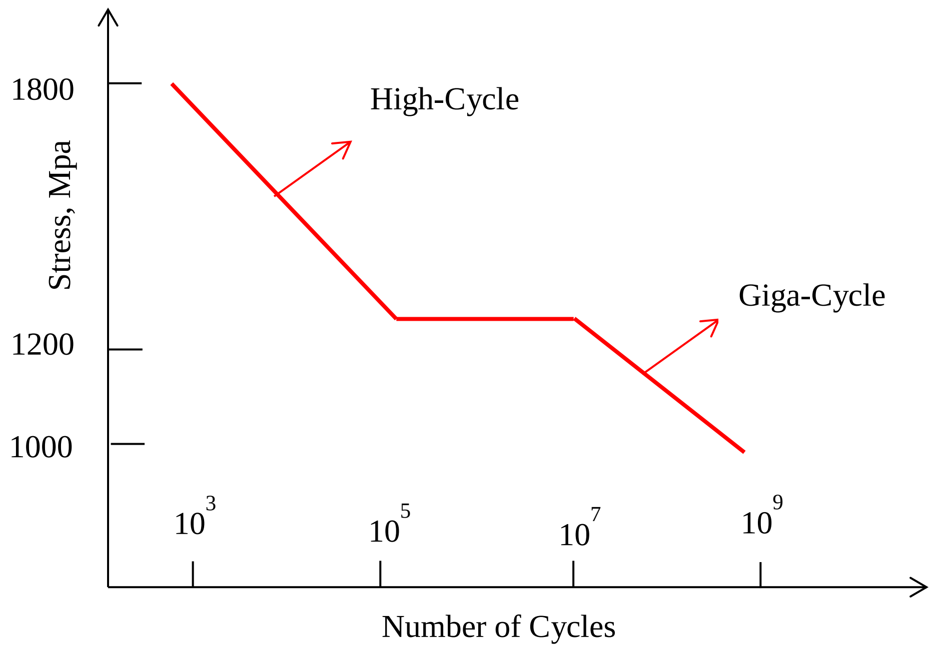


Figure 2.2: Gigacycle S-N Curve (Bathias and Paris 2005).

<u>Prestressed Concrete Bridge Fatigue</u>

An investigation on the fatigue behavior of pretensioned concrete girders was conducted by Overman et al. (1984). This study included an extensive literature review and full-scale fatigue tests of flexural prestressed concrete girders. In addition to the behavior of the whole girders, the fatigue behaviors of the girder components such as the concrete, steel rebars and prestressing strands, as well as the interaction between these materials were discussed. According to a study by the American Concrete Institute (ACI) Committee 215 (ACI 1974), progressive cracking may occur in concrete and fatigue failure may occur after a certain number of repetitive loadings even when the maximum stress of the repetitive loadings is less than the concrete's static strength. In the ACI-215 study (ACI 1974), concrete fatigue strength was determined as a fraction of the concrete static strength.

In the Overman's study, it was found that among the different fatigue failure mechanisms of prestressed concrete girders, the most common fatigue failure was the prestressing strands fatigue fracture (Overman et al. 1984). Especially when cracks occurred in prestressed girders, strands fatigue was more likely to occur at cracked locations because of increased stress range in strands at these cracked locations.

To estimate the prestressing strands fatigue life, the following equation by Paulson et al. (1983) can be used:

$$\mathrm{Log}\ \mathrm{N} = 11 - 3.5 \times \mathrm{Log}\ \sigma \qquad (2.3)$$

where

N: fatigue life in number of stress cycles

σ: prestressing strands stress range in ksi

In this study (Paulson et al. 1983), a literature review of more than 700 seven-wire prestressing strand fatigue test specimens, which included tests conducted in the U.S. and Europe, and new prestressing strands fatigue test results of more than 60 new specimens were both provided. Through regression analysis, data from both the literature review and fatigue test were used to calibrate Equation (2.3). Although minimum stress was found to have an influence on fatigue strength, it was deemed not important enough to be included in this equation (Paulson et al. 1983). Note that this equation was a lower bound relationship equation and there was 95% probability that more than 97.5% data points could be conservatively represented by this equation (Paulson et al. 1983). Similar to the reinforcement steel Equation (2.2), a limiting stress range of 20 ksi was recommended. Paulson et al. (1983) recommended the use of AASHTO Category B redundant structures (AASHTO 2007) design provision for prestressing strand fatigue design when an accurate strand stress range is available.

Overman et al. (1984) pointed out that while Equation (2.3) was derived using the test data of prestressing strands, this equation can also be used to determine the fatigue life of the whole girders with strands embedded in them. The flexural fatigue tests of prestressed girders were also reviewed in their study. Full scale pre-tensioned bridge and post-tensioned bridge fatigue test results from AASHTO were discussed by Overman et al. (1984). In addition, Overman et al. (1984) also conducted new fatigue experiments on 11 flexural pre-tensioned girders. They stated that fatigue life of prestressed concrete girder was primarily governed by the stress range of prestressing strand under repetitive loading and the initiating failure of girder was caused by fatigue fracture of individual wires. Finally, Overman et al. (1984) recommended further research on other factors such as concrete section crack, prestressing loss and overload in order to get a more accurate estimate of the structure fatigue life. Inclusion of these factors usually results in a larger stress range, which might cause a significant reduction in the structure fatigue life (Overman et al. 1984).

AASHTO LRFD Fatigue Specification

AASHTO LRFD specification provides a design fatigue truck with a gross vehicle weight of 54kips and front axle spacing of 14ft and rear axle spacing of 30ft (AASHTO 2007). The design fatigue truck is not meant for representing any particular truck types. It is developed for design purpose based on a distribution of truck weights and truck axle configurations to capture the fatigue loading effect caused by truck traffic. If the truck weight and frequency distribution information are available for a specific site, the gross weight of an equivalent design fatigue truck can be calculated from the following equation (Chotickai and Bowman 2006b).

$$\mathrm{W} = \left(\sum \alpha_i W_i^3\right)^{1/3} \tag{2.4}$$

where

W_i: vehicle gross weight

α_i: frequency of occurrence of trucks

To improve the accuracy of fatigue damage prediction, Chotickai and Bowman (2006a) also suggested the use of Equation (2.4) in lieu of the AASHTO fatigue truck.

For multi-lane bridges, Equation (2.5), which is taken from the AASHTO LRFD specification (AASHTO 2007), can be used to estimate the single-lane average daily truck traffic ($ADTT_{SL}$).

$$ADTT_{SL} = p \times \mathrm{ADTT} \tag{2.5}$$

where p is the fraction of truck traffic for one truck lane, as listed in Table 2.1 (AASHTO 2007) and ADTT is the average daily truck traffic in one direction.

Table 2.1: Fraction of Truck Traffic in a Single Lane (AASHTO 2007).

Number of lanes available to trucks	p
1	1.00
2	0.85
3 or more	0.80

AASHTO LRFD also states that the maximum design ADT (average daily traffic) under normal conditions is limited to around 20000 vehicles per lane (AASHTO 2007). This maximum design ADT can be used to estimate the $ADTT_{SL}$, by multiplying it with the fraction of truck traffic shown in Table 2.2 (AASHTO 2007).

Table 2.2: Fraction of Truck Traffic (AASHTO 2007).

Highway Classification	Fraction of trucks in traffic
Rural Interstate	0.20
Urban Interstate	0.15
Other Rural	0.15
Other Urban	0.10

Overweight Trucks and Bridge Fatigue

Overweight truck loading is one of the greatest concerns to many state departments of transportation. The presence of overweight trucks means load demands may be greater than the design loads, which not only compromises the safety of bridges but may also cause accelerated bridge deterioration. Because overweight trucks could produce a higher stress range, they could significantly reduce the service life of the bridge or even cause fatigue failure. The impact of overloading is more significant for existing bridges because corrosion and other deteriorations may already have occurred in existing bridges due to years of exposure to deicing agents and environmental elements (Jaffer and Hansson 2009). The occurrence of cracks combined with overweight trucks would result in higher stress ranges and ultimately reduces the bridge fatigue life.

An Indiana study (Chotickai and Bowman 2006a) evaluated the steel bridge fatigue damage caused by overweight vehicles along a high traffic volume highway in Northern Indiana. Weigh-in-motion (WIM) system was used to get the truck weight distribution. The FHWA Class 9 (FHWA 2013) trucks and Class 13 trucks were found to be the two most common truck types (Chotickai and Bowman 2006a). The maximum weights for these two types of trucks were 150,000 lbs and 200,000 lbs, respectively (Chotickai and Bowman 2006a). Average truck gross weight for all trucks in all directions on this highway was 52,368 lbs (Chotickai and Bowman 2006a). Class 9 truck had an average gross weight of 54,356 lbs and Class 13 trucks had an average weight of 119,459 lbs (Chotickai and Bowman 2006a). Strain gages were installed to obtain strain range and to estimate fatigue damage. According to Chotickai and Bowman (2006a), fatigue failure was not a concern for the bridges in Indiana because overweight trucks, which could cause significant fatigue damage, made up less than 1% of the whole truck population in Indiana (Chotickai and Bowman 2006a).

In a recent study of steel and prestressed concrete bridge fatigue damage caused by increased truck weight performed by the University of Minnesota (Altay et al. 2003), researchers selected five steel bridges and three prestressed concrete bridges on Minnesota highway for instrumentation and loading. For comparison purpose, the selected bridges were also modeled using the SAP2000 software and the remaining fatigue lives were calculated for all eight bridges. They found that for prestressed concrete bridges, a 10% to 20% increase in allowable gross vehicle weight did not have a significant impact on the fatigue life of bridges because of a very small increase in the stress range (Altay et al. 2003). In fact, the analyses results showed that prestressed bridges have infinite fatigue lives. For most modern steel bridges, a 20% increase in truck weight would not cause fatigue issue. However, for certain steel bridges with very high traffic volumes and very poor fatigue details, fatigue might be a safety concern (Altay et al. 2003).

Overweight Trucks and Bridge Cost

One study from Ohio Department of Transportation computed the annual bridge cost and the portion of cost associated with overweight vehicles (ODOT 2009). They calculated the total bridge asset value from the current replacement cost of all bridges in Ohio and by assuming 1/75 of this cost is consumed each year (i.e. based on the target bridge design life of 75 years specified in AASHTO). In addition, the annual bridge preservation or maintenance cost was also computed. The total annual bridge cost, including both the damage and maintenance costs, in Ohio was found to be approximately $308 million dollars (ODOT 2009).

In the ODOT study, the annual bridge asset value was allocated to overweight vehicles using a methodology called the *incremental cost analysis* (ODOT 2009). In the incremental cost analysis, a bridge was designed using the full design load and its cost was calculated. Then a group of heaviest vehicles were removed from the calculation of the design load. The bridge was redesigned using a lower design load and a new cost was computed. The differences between these two costs were then assigned to the heavier vehicle group. By repeating this process, they were able to allocate the cost to overweight vehicles and other vehicles (ODOT 2009). For the annual bridge preservation or maintenance cost, the cost associated with overweight vehicles were allocated using the vehicle miles traveled (VMT) ratio of overweight vehicles, as a fraction of the total truck VMT (ODOT 2009). Adding up the annual bridge asset value of overweight vehicles and annual bridge preservation cost of overweight vehicles, they found the annual bridge cost associated with overweight vehicles in Ohio to be approximately $22 million dollars (ODOT 2009).

CHAPTER THREE

METHODOLOGY CHART

Methodology Chart Development

The methodology used in this research to calculate the annual bridge cost is shown in Figure 3.1. A brief discussion about the methodology developed to determine the total bridge cost (including both damage and routine maintenance costs), is provided in the next section and more details for each modules shown in Figure 3.1 are discussed in Chapters 4 to 11.

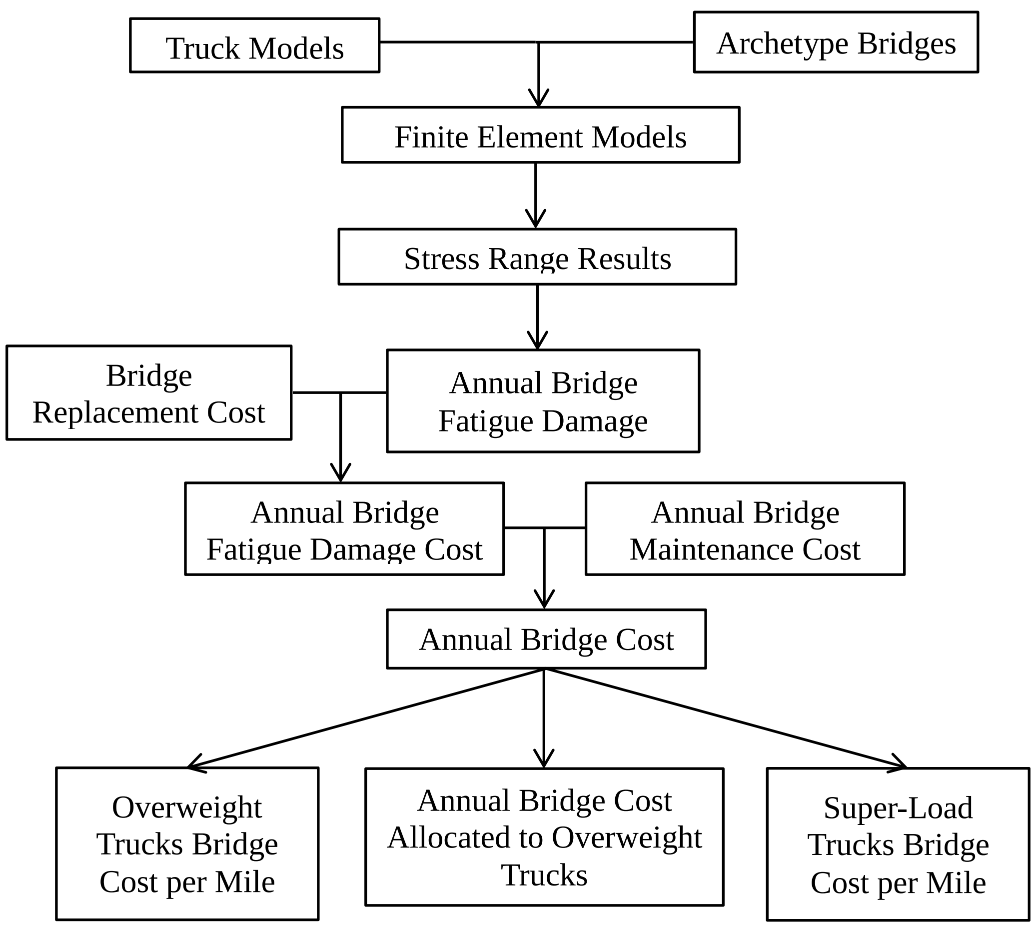


Figure 3.1: Methodology.

Methodology for Determining Bridge Cost

The main objective of this research was to determine the annual bridge cost and cost associated with overweight trucks in South Carolina. The first step was to develop a series of representative truck models to represent the truck population in South Carolina. These truck models were developed based on the truck gross weight distribution, truck axle configuration distribution, and truck weight limits in South Carolina.

Due to the large number of bridges in South Carolina (9,271 bridges), it was not feasible to create a finite element (FE) model for each bridge. The second step was to develop *Archetype bridges* to represent group of bridges which share common features and structural characteristics. Bridge information such as the material, span length, count, location and etc. were obtained from the NBI database (NBI 2012).

The third step was to build finite element (FE) models for all Archetype bridges using a finite element program, called LS-DYNA. In this step, the FE models were developed and analyzed with different combinations of Archetype bridges and truck models (with different truck weights and axle configuration).

The fourth step was to solve the finite element models built in the third step and to record the stress ranges for each analysis.

The fifth step was to quantify the annual bridge fatigue damage for all Archetype bridges using stress ranges calculated form the FE analysis.

In order to estimate the damage costs caused by truck traffic on bridges, the replacement costs of individual bridges were determined at the sixth step.

With bridge replacement cost and annual bridge fatigue damage determined, the seventh step was to calculate the annual bridge cost. This annual bridge cost included two parts: bridge fatigue damage cost and bridge maintenance cost.

Finally, the annual bridge cost allocated to overweight trucks was calculated based on the overweight truck damage contribution and the percentage of overweight trucks in total truck population. In addition to compute the damage cost contribution of overweight trucks, unit costs (cost per mile) of individual truck types of different axle configurations and gross weights were computed using the vehicle miles traveled (VMT) of individual truck types.

CHAPTER FOUR

TRUCK MODEL DEVELOPMENT

Introduction

In order to estimate fatigue damage caused by trucks with different weights and axle configurations, truck models representative of the truck population were developed based on truck gross weight distribution, truck axle configuration distribution, and truck weight limits in South Carolina.

Truck Model Development

According to the number of axles, trucks were grouped into 7 axle groups. The percent of trucks for each vehicle class in South Carolina was determined using the weigh-in-motion data for a selected location (StGeorge1) in South Carolina (SCDPS 2012a). Table 4.1 shows the truck distribution recorded at the St George station. Details of weigh-in-motion data (SCDPS 2012a) are provided in Appendix A.

Table 4.1: Vehicle Class Percentage.

FHWA Vehicle Class	Axle Group	Percentage
5	2-Axle	8.84%
6	3-Axle	1.15%
7	4-Axle	0.05%
8	3-Axle	9.10%
	4-Axle	
9	5-Axle	75.97%
10	6-Axle	2.30%
	7-Axle	
11	5-Axle	2.52%
12	6-Axle	0.02%
13	7-Axle	0.06%
	8-Axle	

The mapping between the FHWA vehicle class (FHWA 2013) and axle group is also shown in Table 4.1. Grouping of the truck distribution by axle group is shown in Table 4.2. To group the trucks by axle group, it was assumed that half of the FHWA class 8 trucks were 3 axles and half of them were 4 axles. The same assumption was also applied to the class 10 trucks and class 13 trucks. The percentage of 3-axle trucks is equal to the sum of the percentage of class 6 trucks plus half of the percentage of class 8 trucks (1/2 x 9.10%).

Table 4.2: Truck Axle Group Distribution.

Axle Group	Percentage
2-Axle	8.84%
3-Axle	5.70%
4-Axle	4.60%
5-Axle	78.49%
6-Axle	1.17%
7-Axle	1.18%
8-Axle	0.03%

As seen in Table 4.2, the predominate truck type was 5-axle truck (78.49%) and the least common truck type was 8-axle truck (0.03%).

Three different gross vehicle weights (GVW) were assigned to each axle group to represent the truck weight distribution within each axle group. These gross vehicle weights were determined as:

GVW1: 80% of the SCDOT legal weight limit;

GVW2: SCDOT maximum weight limit;

GVW3: maximum considered truck weight.

The SCDOT legal weight limits for different axle groups were obtained from the SC code of laws (SC Code of Laws 2012) while the SCDOT maximum weight limits were obtained from the SCDOT website (SCDOT 2012a). The *maximum considered truck weight* for each axle group was determined using the maximum observed truck weight in the size and weight inspection violations data provided by the South Carolina Department of Public Safety (SCDPS 2012b) and overweight truck permit data (SCDOT 2012b). More information about the SCDOT overweight truck permit data can be found in Appendix B.

Table 4.3 shows the SCDOT legal weight limits and maximum weight limits. Table 4.4 shows the three levels of GVWs for all axle groups utilized in this study.

Table 4.3: SCDOT Weight Limit (SC Code of Laws 2012) (SCDOT 2012).

Truck	Legal Limit (kips)	Maximum Limit (kips)
Two axle single unit	35	40
Three axle single unit	46	50
Four axle single unit	63.5	65
Three axle combination	50	55
Four axle combination	65	70
Five axle combination	80	90
Six axle combination	80	110
Seven axle combination	80	130
Eight axle combination	80	130

Table 4.4: Truck GVW in Each Axle Group.

Axle Group	GVW1 (kips)	GVW2 (kips)	GVW3 (kips)
2-Axle	28	40	48
3-Axle	40	55	70
4-Axle	52	70	90
5-Axle	64	90	130
6-Axle	64	110	139
7-Axle	64	130	200
8-Axle	64	130	170

The percent of truck associated with each GVW level and axle group shown in Table 4.4 was determined using the weigh-in-motion data. From the weigh-in-motion data, the cumulative counts or numbers of trucks by gross weight for each vehicle class were used to fit the truck distribution to the 3-parameter Weibull distribution. The cumulative distribution function (CDF) for the 3-parameter Weibull distribution is:

$$\mathrm{F(x)} = 1 - \exp\left[-\left(\frac{x-w}{u-w}\right)^{k}\right] \tag{3.1}$$

where

x: truck weight

u: scale parameter (>0)

w: location parameter (lower limit of x, 10 kips was assumed as the base truck weight)

k: shape parameter (>0)

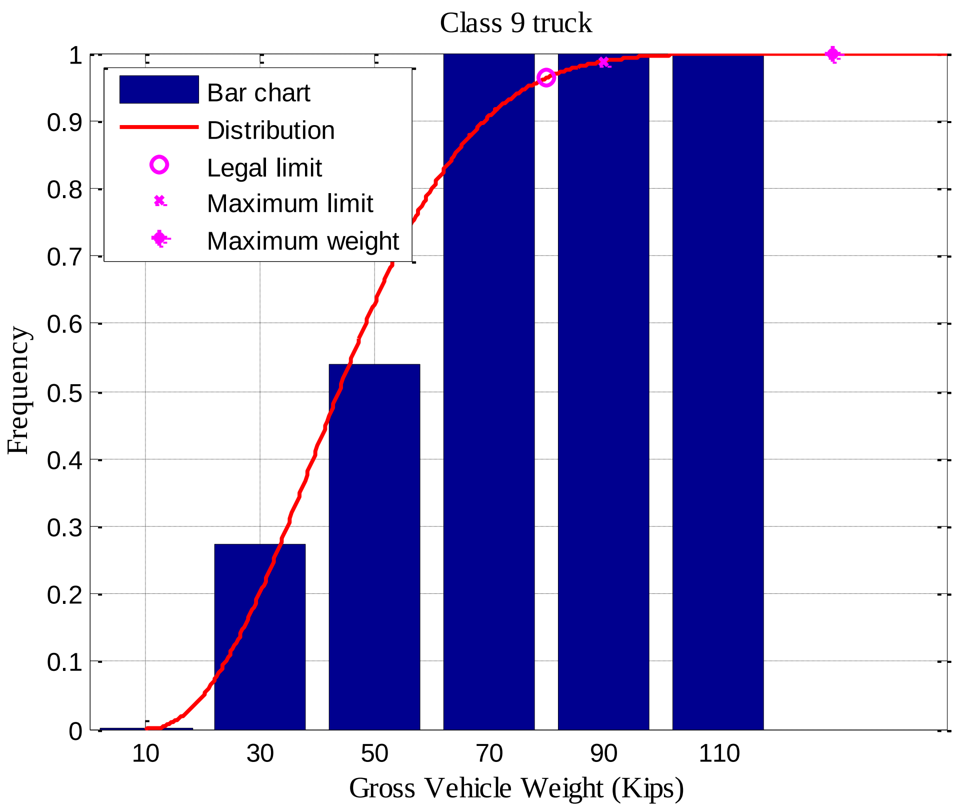


Figure 4.1: Class 9 Truck Weight Distribution Model.

Figure 4.1 shows the cumulative distribution of the class 9 truck determined using the weigh-in-motion data (SCDPS 2012a). The blue bars represent the cumulative percentage of trucks with different gross weights and the red curve represents the fitted distribution model. With the CDF for each vehicle class determined, the probability density function (PDF) for the 3-parameter Weibull distribution was then obtained using the following equation:

$$\mathrm{f(x)} = \frac{k}{u-w}\left(\frac{x-w}{u-w}\right)^{k-1} \exp\left[-\left(\frac{x-w}{u-w}\right)^{k}\right] \quad (3.2)$$

An example of the PDF curve for the class 9 truck is given in Figure 4.2.

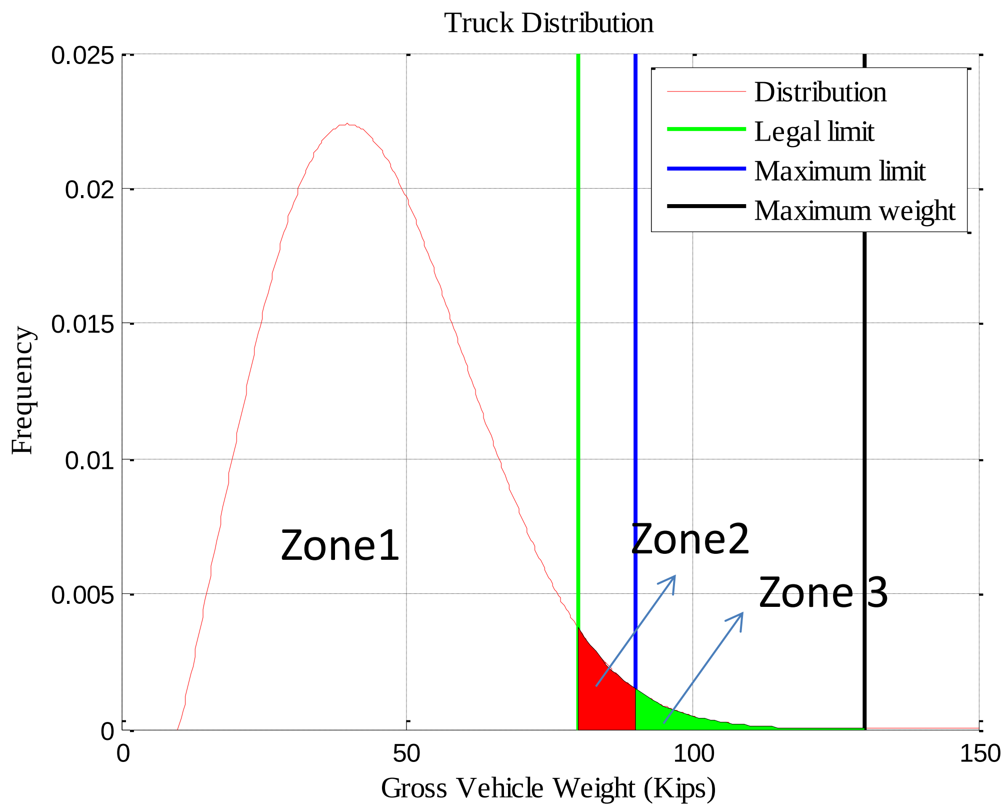


Figure 4.2: Truck Gross Weight Distribution for Vehicle Class 9.

Figure 4.2 shows the PDF curve for the class 9 truck. Zone 1 includes those trucks with their gross vehicle weights less than the legal weight limit. For analysis purpose, the percentage of these trucks (i.e. area of Zone 1) was conservatively assigned to GVW1 (80% of the SCDOT legal weight limit). Zone 2 represents the percentage of trucks with gross vehicle weights between the legal limit and the maximum limit (see Table 4.4). The area of Zone 2 was assigned to GVW2 (SCDOT maximum weight limit). Similarly, Zone 3 represents the trucks with gross vehicle weights larger than the maximum limit and this percentage was assigned to GVW3 (maximum considered truck weight). The percent distributions of GVW1 to GVW3 for all vehicle classes are given in Table 4.5. Details of the fitted gross vehicle weight distribution parameters and figures for all vehicle classes are shown in Appendix A.

Table 4.5: Gross Vehicle Weight Distribution by Vehicle Class.

FHWA Vehicle Class	Axle Group	Percentage of GVW1	Percentage of GVW2	Percentage of GVW3
5	2-Axle	99.98%	0.01%[a]	0.01%[a]
6	3-Axle	99.90%	0.08%	0.02%
7	4-Axle	99.91%	0.08%	0.01%[a]
8	3-Axle	99.92%	0.06%	0.02%
	4-Axle	99.98%	0.01%[a]	0.01%[a]
9	5-Axle	92.68%	4.82%	2.50%
10	6-Axle	95.86%	4.08%	0.06%
	7-Axle	95.85%	4.14%	0.01%[a]
11	5-Axle	99.95%	0.04%	0.01%[a]
12	6-Axle	75.00%	23.61%	1.40%
13	7-Axle	32.98%	54.20%	12.82%
	8-Axle	32.98%	54.20%	12.82%

(a) Note that some of the cells had zero observations. This is because the weigh-in-motion data were collected for one location (StGeorge1) over a six-month period. For those GVW2 and GVW3 cells with zero observations, a nominal percentage of 0.01% was assumed to consider the unaccounted overweight trucks due to the limited data.

Using the mapping between the FWHA vehicle class and axle groups shown in Table 4.1, the gross vehicle weight distribution by vehicle class (Table 4.5) was then grouped by the number of axles and the results are shown in Table 4.6. As seen in Table 4.6 and as expected, there are very few GVW2 and GVW3 trucks recorded in 2-axle, 3-axle and 4-axle trucks.

Table 4.6: Gross Vehicle Weight Distribution by Axle Group.

Axle Group	Percentage of GVW1	Percentage of GVW2	Percentage of GVW3
2-Axle	99.98%	0.01%	0.01%
3-Axle	99.92%	0.06%	0.02%
4-Axle	99.98%	0.01%	0.01%
5-Axle	92.91%	4.66%	2.42%
6-Axle	95.54%	4.38%	0.08%
7-Axle	94.25%	5.41%	0.34%
8-Axle	32.98%	54.20%	12.82%

In addition to gross vehicle weight and number of axles, bridge damage may also be affected by the spacing of axles. For instance, one might expect a truck with closely spaced axles to be more damaging to bridges than a truck with the same weight but with axles spaced further apart. In order to account for the influence of axle configuration (i.e. axle spacing) on bridge damage, information on the axle spacing was incorporated into the surrogate truck models. The truck axle configuration information (axle spacing, axle weight) associated with each truck weight was determined from the SCDOT overweight truck permit data (SCDOT 2012b). Since GVW1 and GVW2 trucks consisted of the majority of the trucks within each axle group, the most common truck axle configuration recorded in the SCDOT overweight truck permit data was assigned to GVW1 and GVW2 trucks. Since the GVW3 was derived using the maximum gross weight recorded in the SCDOT truck permit data (SCDOT 2012b) and the size and weight inspection violations data (SCDPS 2012b), the axle configuration corresponded to the particular truck with the highest observed weight in the permit data was used for GVW3 truck. Therefore, the configuration (axle spacing) of the GVW3 truck model for each axle group might not be the same as that of GVW1 and GVW2. For instance, for the 4-axle trucks, there were three common axle configurations recorded in the SCDOT overweight truck permit data. All three axle configurations were selected to represent the 4-axle group. Table 4.7 shows the axle spacing for each truck type and Table 4.8 presents the weight of each truck axle for each truck type. As can be seen from the these two tables, except for the 4-axle and 2-axle trucks, there were three different GVWs and two types of axle configurations for each axle group; hence three truck models were developed to represent the trucks in each axle group. For the 4-axle trucks, 9 truck models were developed. A total of 27 truck models were developed to represent the whole truck population.

Table 4.7: Truck Axle Spacing Configuration.

Axle Group	Truck Type	Distance 1st axle-2nd axle (ft)	Distance 2nd axle-3rd axle (ft)	Distance 3rd axle-4th axle (ft)	Distance 4th axle-5th axle (ft)	Distance 5th axle-6th axle (ft)	Distance 6th axle-7th axle (ft)	Distance 7th axle-8th axle (ft)
2-Axle	A21	20						
3-Axle	A31	20	5					
	A32	15	5					
4-Axle	A41	15	5	42				
	A42	4	15	5				
	A43	4	23	4				
	A44	17	30	5				
	A45	17	37	4				
5-Axle	A51	14	5	60	5			
	A52	17	4	37	5			
6-Axle	A61	11	5	25	4	4		
	A62	17	5	36	5	5		
7-Axle	A71	5	5	10	5	8	5	
	A72	12	4	4	36	5	5	
8-Axle	A81	16	5	5	24	9	8	5
	A82	12	4	4	35	5	5	11

Table 4.8: Truck Axle Weight Configuration.

Axle Group	Truck Type	Axle Weight of GVW1 (kip)	Axle Weight of GVW2 (kip)	Axle Weight of GVW3 (kip)
2-Axle	A21	14+14	20+20	24+24
3-Axle	A31	N/A	N/A	20+25+25
	A32	12+14+14	17+19+19	N/A
4-Axle	A41	10+13+13+16	13+18+18+21	22+22+23+23
	A42	N/A	N/A	22+22+23+23
	A43	12+12+14+14	15+15+20+20	N/A
	A44	N/A	N/A	22+22+23+23
	A45	10+16+13+13	12+22+18+18	N/A
5-Axle	A51	N/A	N/A	12+17+17+42+42
	A52	8+14+14+14+14	14+19+19+19+19	N/A
6-Axle	A61	N/A	N/A	11+31+31+22+22+22
	A62	7+12+12+12+12+9	12+20+20+20+20+18	N/A
7-Axle	A71	N/A	N/A	26+29+29+29+29+29+29
	A72	4+10+10+10+10+10+10	10+20+20+20+20+20+20	N/A
8-Axle	A81	N/A	N/A	9+23+23+23+23+23+23+23
	A82	3+7+9+9+9+9+9+9	12+16+17+17+17+17+17+17	N/A

CHAPTER FIVE

ARCHETYPE BRIDGES DEVELOPMENT

Introduction

According to the National Bridge Inventory database (NBI 2012), maintained by the Federal Highway Administration, there are 9,271 bridges in the state of South Carolina (SC). Due to the large number of bridges, it was not feasible to create a finite element model for each bridge. For modeling purpose, these bridges were grouped into Archetypes. Each Archetype bridge model was used to represent a group of bridges sharing common features and structural characteristics. To facilitate the development of Archetype models, bridge information such as the material, span length, count, location and etc. was obtained from the NBI database. Tables 5.1 to 5.4 show the distribution of bridges in SC categorized by construction materials, structural systems, number of span, and maximum span length, respectively.

National Bridge Inventory Information

Table 5.1: Distribution of SC bridges Based on Construction Materials.

Description	Count	Percentage
1 Concrete	5,028	54.23%
2 Concrete continuous	533	5.75%
3 Steel	948	10.23%
4 Steel continuous	389	4.20%
5 Prestressed concrete	2,014	21.72%
6 Prestressed concrete continuous	261	2.82%
7 Wood or Timber	82	0.88%
8 Masonry	4	0.04%
9 Aluminum, Wrought Iron, or Cast Iron	10	0.11%
0 Other	2	0.02%
Total	9,271	

Table 5.2: Distribution of SC bridges Based on Structure Systems.

Description	Count	Percentage
01 Slab	4,297	46.35%
02 Stringer/Multi-beam or Girder	2,847	30.71%
03 Girder and Floorbeam System	17	0.18%
04 Tee Beam	850	9.17%
05 Box Beam or Girders - Multiple	30	0.32%
06 Box Beam or Girders - Single or Spread	9	0.10%
07 Frame (except frame culverts)	5	0.05%
08 Orthotropic	0	0.00%
09 Truss - Deck	0	0.00%
10 Truss - Thru	37	0.40%
11 Arch - Deck	48	0.52%
12 Arch - Thru	0	0.00%
13 Suspension	0	0.00%
14 Stayed Girder	1	0.01%
15 Movable - Lift	0	0.00%
16 Movable - Bascule	3	0.03%
17 Movable - Swing	5	0.05%
18 Tunnel	2	0.02%
19 Culvert (includes frame culverts)	1,086	11.71%
20 * Mixed types	0	0.00%
21 Segmental Box Girder	2	0.02%
22 Channel Beam	20	0.22%
00 Other	12	0.13%
Sum	9,271	

Table 5.3: Distribution of SC bridges Based on Number of Spans.

Description	Count	Percentage
1	1,625	17.53%
2	1,638	17.67%
3	2,549	27.49%
4	1,347	14.53%
5	825	8.90%
6	384	4.14%
7	212	2.29%
8	210	2.27%
9	76	0.82%
10	90	0.97%
11	49	0.53%
12	43	0.46%
13	35	0.38%
14	20	0.22%
15	25	0.27%
16	19	0.20%
17	17	0.18%
18	11	0.12%
Else	96	1.04%
Sum	9,271	

Table 5.4: Distribution of SC bridges Based on Maximum Span.

Description	Count	Percentage
<5m	3,696	39.87%
5m-10m	2,447	26.39%
10-15m	828	8.93%
15m-20m	960	10.35%
20m-25m	494	5.33%
25m-30m	270	2.91%
Else	576	6.21%
Sum	9,271	

Archetype Bridge Development

As can be seen from Table 5.1, reinforced concrete, prestressed concrete and steel are the three main construction materials which account for more than 98% of all bridges in SC. Table 5.2 shows that slab and stringer/multi-beam or multi-girder are the two most commonly used structure systems for the superstructure. From Tables 5.3 and 5.4, one can observe that approximately 77% of all the bridges are with four or less spans (Table 5.3) and the maximum span length for most of the bridges are less than 20 meters (66 ft) (Table 5.4). Considering all the above information and due to time constraint, four Archetype bridges were selected as surrogate bridge models and analyzed in this study (Table 5.5).

Table 5.5: Archetype Bridges.

Archetype	Description
1	Reinforcement concrete slab bridge with span of 10m (33ft)
2	Prestressed concrete beam bridge with span less than 20m (66ft)
3	Prestressed concrete beam bridge with span 20m (66ft) to 35m (115ft)
4	Prestressed concrete beam bridge with span 35m (115ft) to 45m (148ft)

Detailed drawings for selected as-built bridges suitable for the four Archetype bridges were obtained from the SCDOT and used to develop the FE bridge models. More details on the drawings and structural systems of these bridges are discussed in the following paragraphs.

A set of standard structural drawings for Archetype 1 bridge was obtained from the SCDOT website (SCDOT 2011). SCDOT provides standard drawings for slab bridges of span length of 30ft, 60ft, and up to 120ft. The structural drawings for the 30ft span superstructure with 34ft roadway were used to develop the finite element model for Archetype 1 bridge (Figure 5.1 and Figure 5.2).

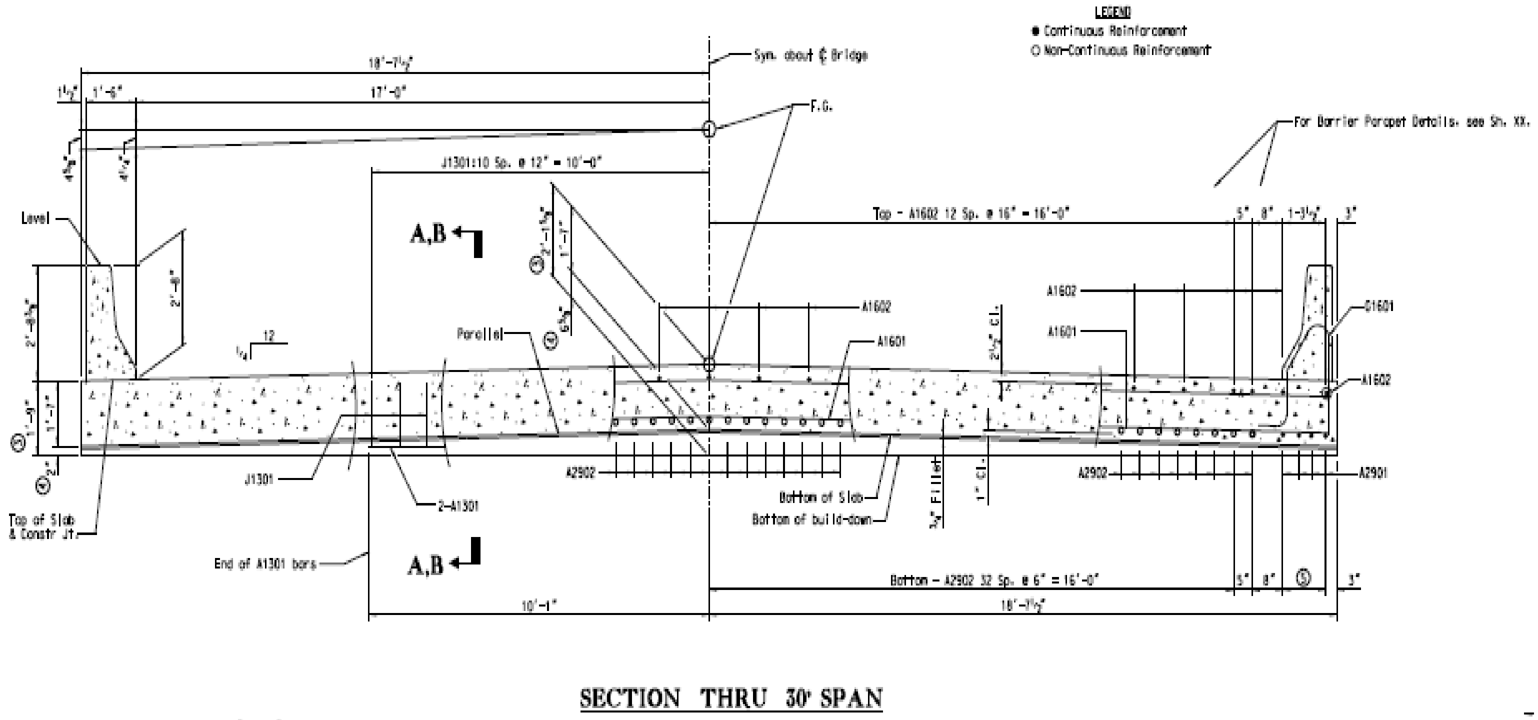


Figure 5.1: Cross-Sectional View of Archetype 1 Bridge (SCDOT 2011).

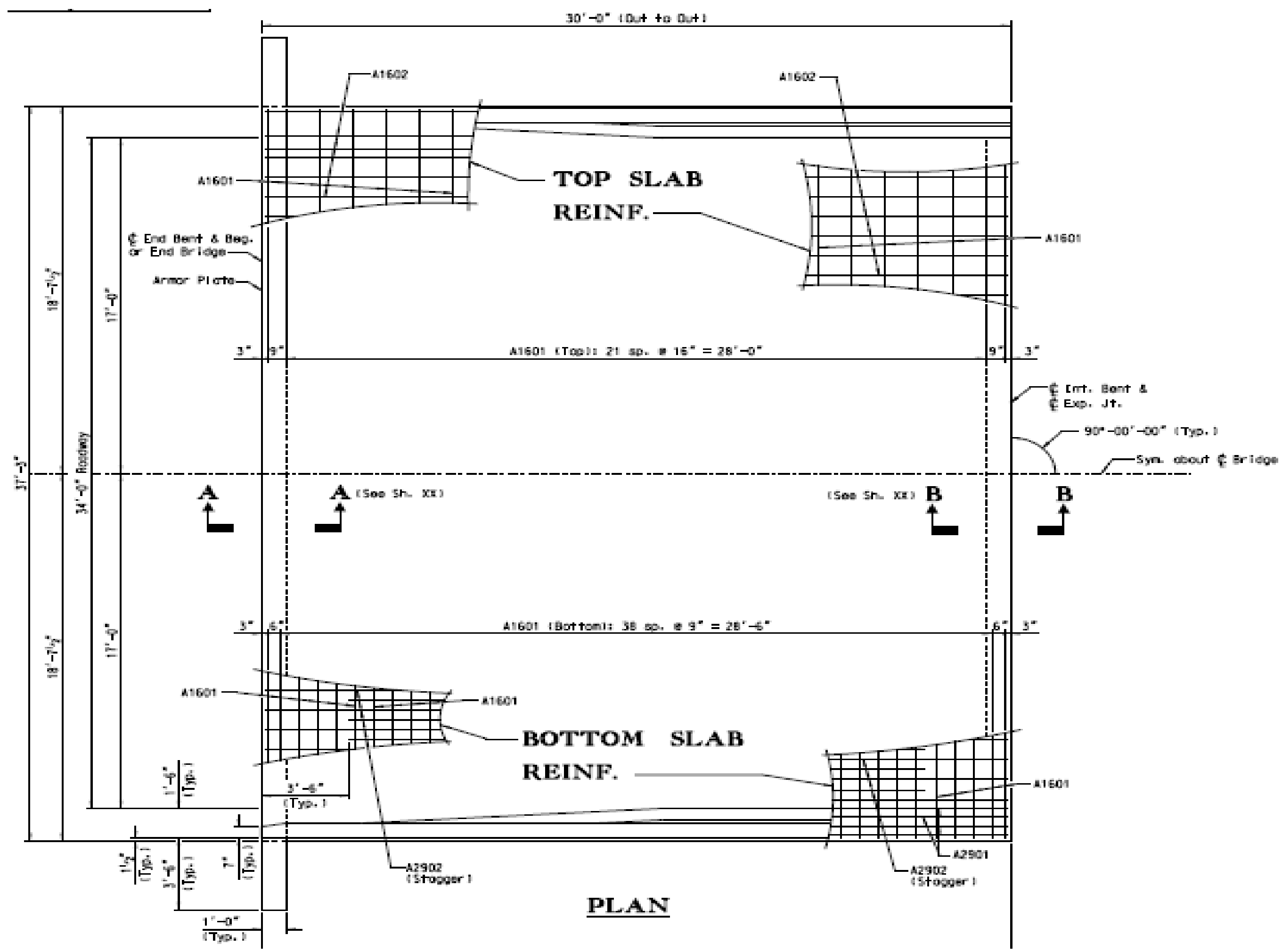


Figure 5.2: Plan View of Archetype 1 Bridge (SCDOT 2011).

For Archetype 2 and Archetype 3 bridges, the structural drawings of a simply supported prestressed concrete dual overpass girder bridge located at the Marshland Road, Beaufort County were selected as the reference drawings. The as-built bridge drawings (SCDOT bridge reference number 7.581.3) were obtained from the SCDOT (Barrett 2011). This bridge has three spans. On the southbound, the middle span length is 84 ft and 6 in. long and the side span length is 45 ft. On the northbound, the middle span length is 84 ft 6 in long and the side span length is 41 ft and 3 in. Bridge width is 40 feet 10 in. and roadway width is 38 ft. The structural configuration of the bridge side span on the southbound, which is the 45 feet span, was adopted to develop the finite element model for Archetype 2 bridge. The structural configuration of the bridge middle span on the southbound, which is the 84 ft 6 in span, was adopted for modeling Archetype 3 bridge. Figure 5.3 to figure 5.5 show the details.

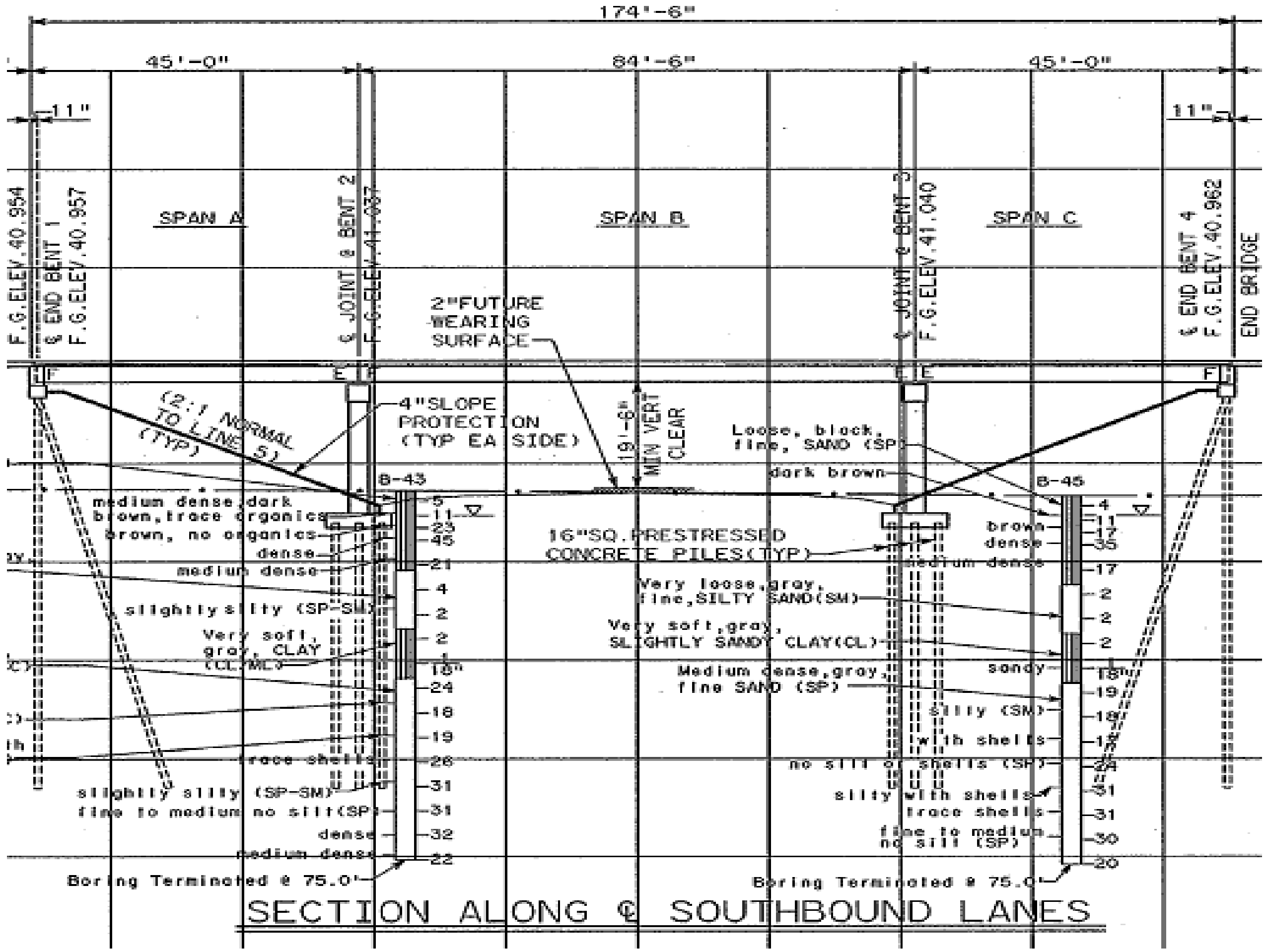


Figure 5.3: Elevation View of Archetype 2 and 3 Bridges (Barrett 2011).

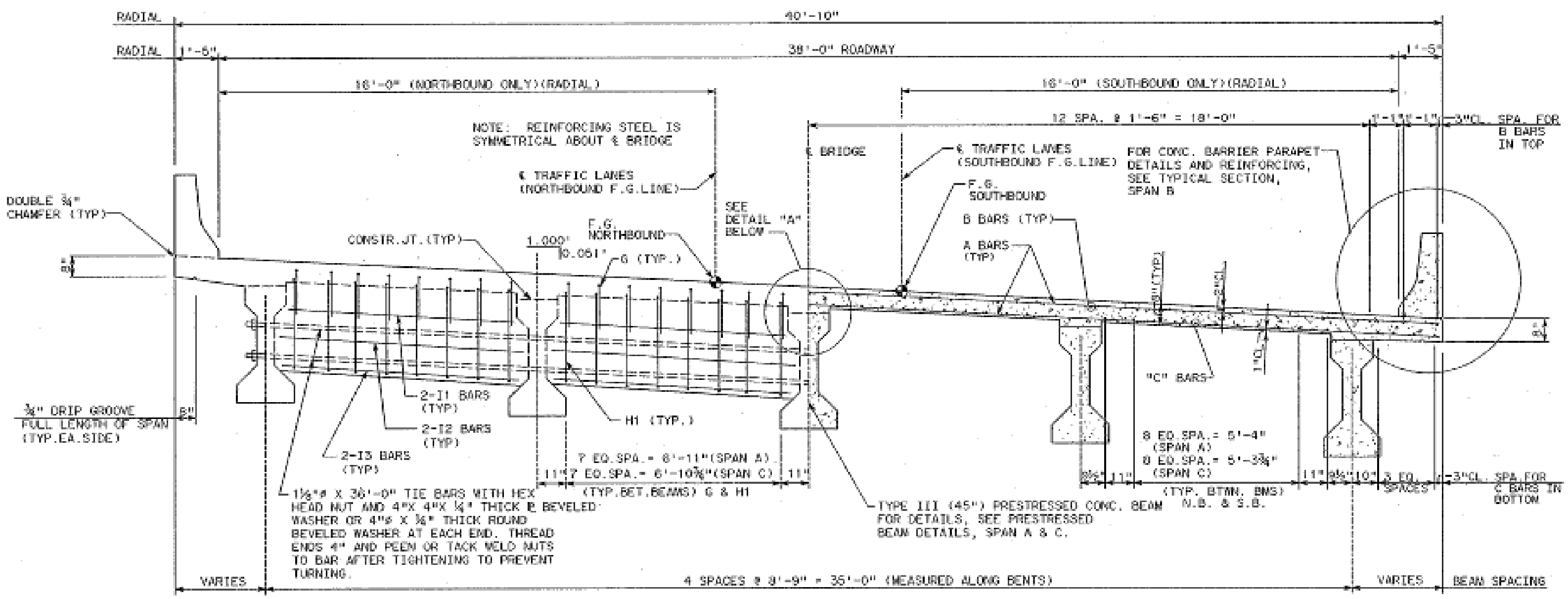


Figure 5.4: Cross-Sectional View of Archetype 2 Bridge (Barrett 2011).

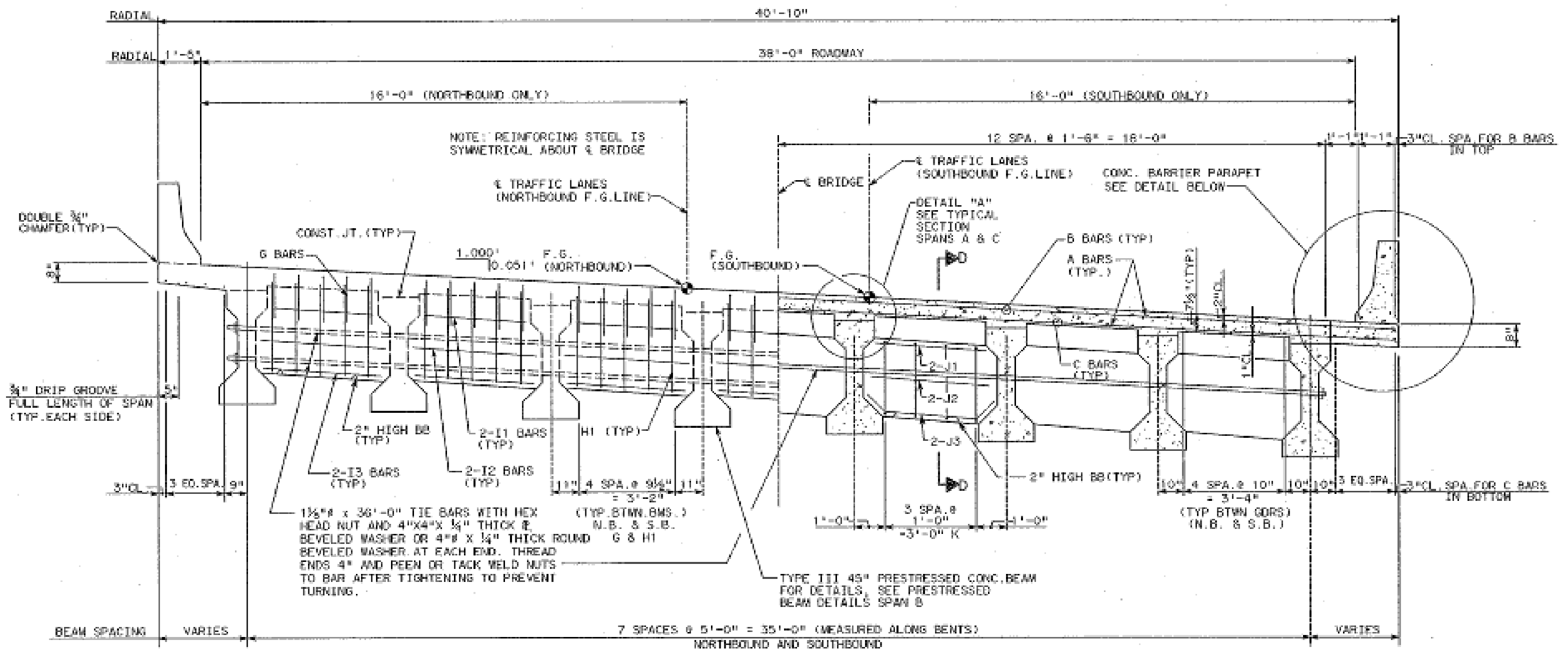


Figure 5.5: Cross-Sectional View of Archetype 3 Bridge (Barrett 2011).

For Archetype 4 bridge, the structural drawings (SCDOT bridge reference number 19.103B) of a simply supported prestressed concrete girder bridge over the Horne Creek, at Edgefield county were used to develop the FE model. These drawings were obtained from SCDOT (Barrett 2012). This bridge has two spans. Each span is 120 ft. The bridge width is 46 ft 10 in. and the roadway width is 44 ft (see Figure 5.6 and Figure 5.7).

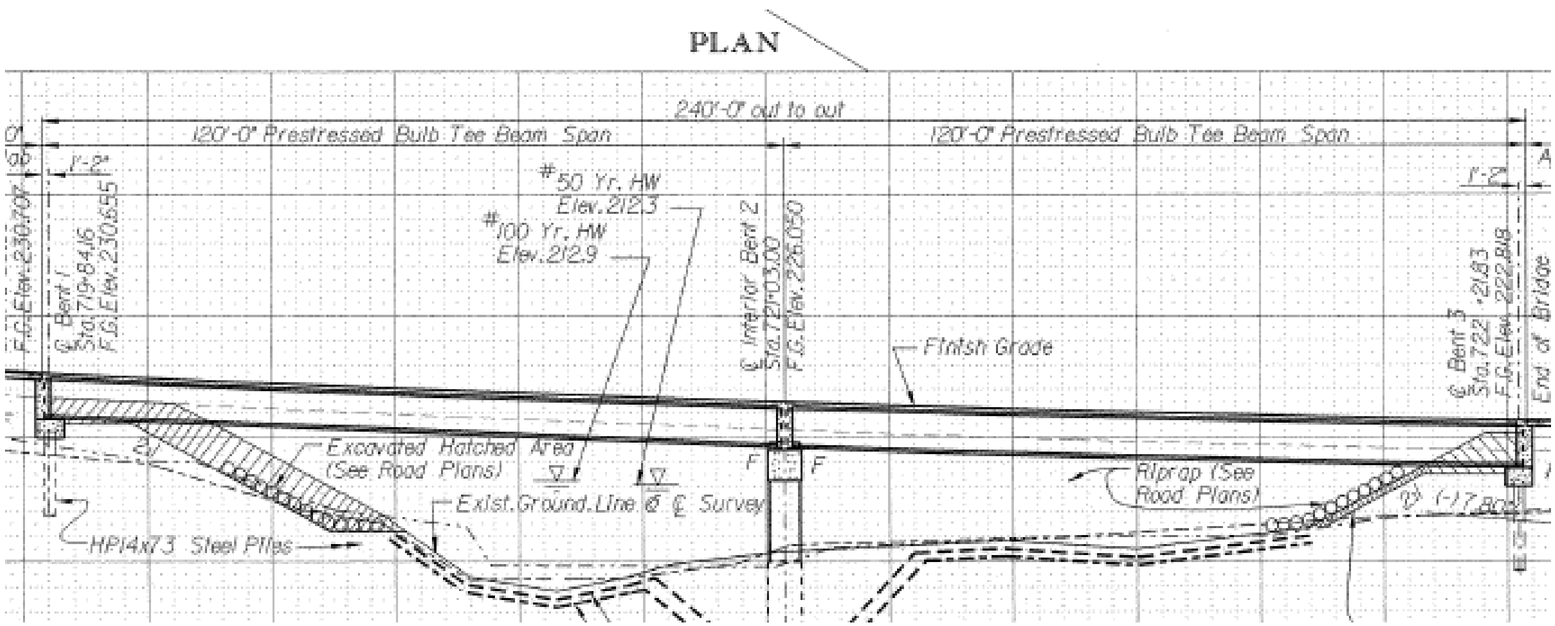


Figure 5.6: Elevation View of Archetype 4 Bridge Drawing (Barrett 2012).

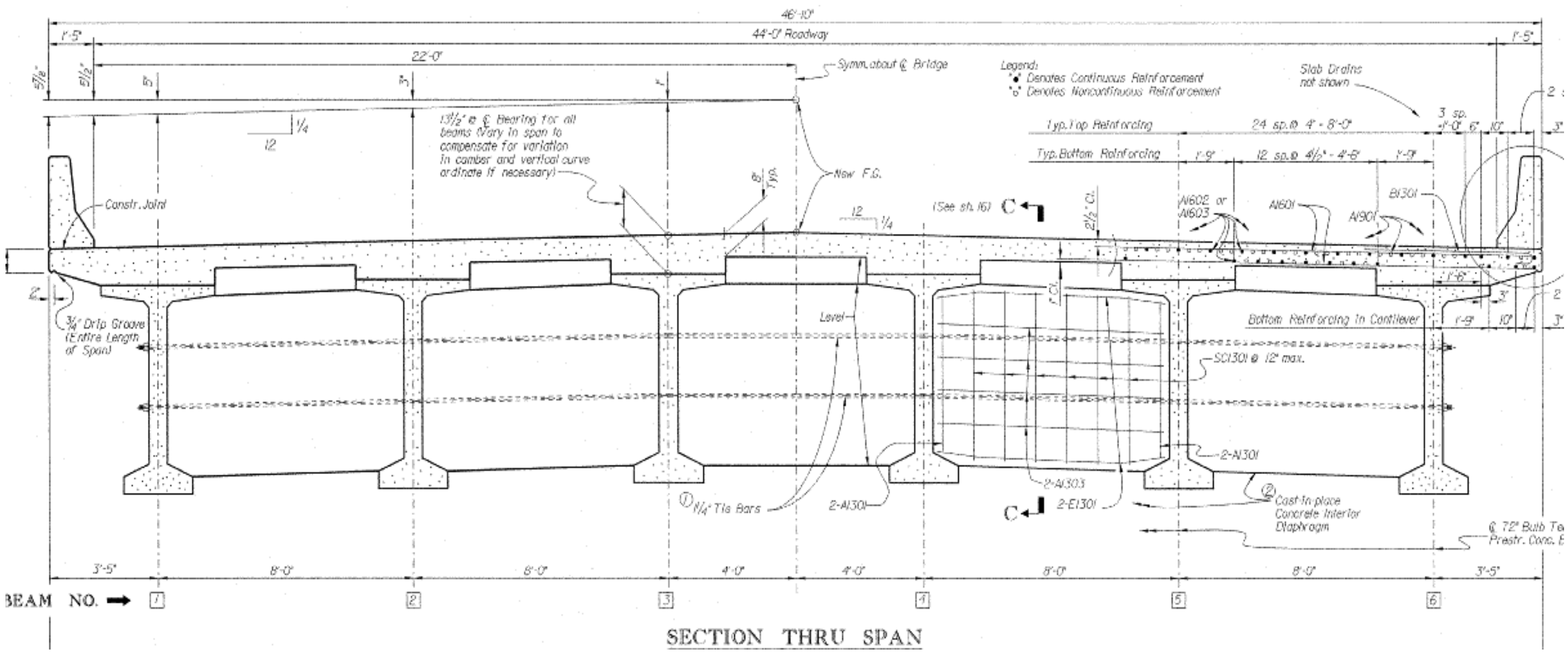


Figure 5.7: Cross-Sectional View of Archetype 4 Bridge Drawing (Barrett 2012).

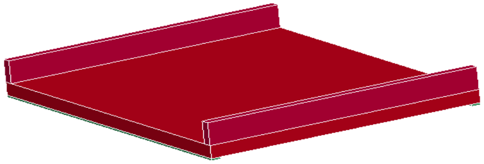
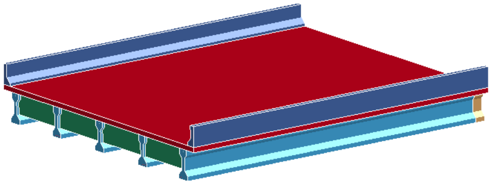
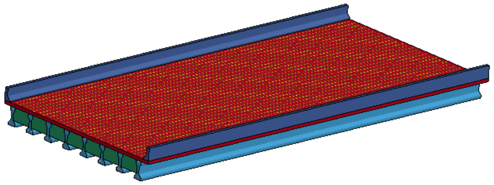
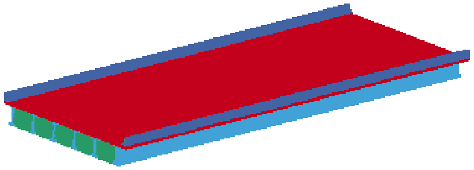
CHAPTER SIX

FINITE ELEMENT MODELS AND RESULTS

Introduction

The structural behavior of Archetype bridges was analyzed using the LS-DYNA finite element (FE) analysis program. LS-DYNA software is a very versatile FE program, which can be used to accurately capture the dynamic responses of bridges under the movement of truck traffic and it can give more accurate stress results than the static analysis (Wekezer et al. 2010). Due to high computational demand of the FE bridge models, the finite element analyses were performed using the Argonne National Laboratory supercomputer. Four Archetype bridges with truck models were first modeled using the LS-PREPOST software and then solved using the LS-DYNA program. The LS-PREPOST is a very powerful preprocessor for the LS-DYNA program. Boundary conditions, material properties, loadings, contact information between tires and bridge slabs and all other necessary information were defined in the LS-PREPOST. The LS-PREPOST was also used as a postprocessor to view the analysis results. The four Archetype bridges are shown in Table 6.1. The details of the four Archetype bridge models and analysis results are discussed in following sections.

Table 6.1: Archetype Bridge Models Summary

Archetype	Description	Models
1	Reinforcement concrete slab bridge with span of 10m (33ft)	
2	Prestressed concrete beam bridge with span less than 20m (66ft)	
3	Prestressed concrete beam bridge with span 20m (66ft) to 35m (115ft)	
4	Prestressed concrete beam bridge with span 35m (115ft) to 45m (148ft)	

Archetype 1 Bridge

Figure 6.1 to Figure 6.3 show the finite element model of the Archetype 1 bridge. The concrete slab was modeled using the fully integrated 3-D 8-node solid elements. The default setting for the 3D 8-node elements in LS-DYNA is one integration point. Using the fully integrated solid element takes more computation time than the element with only one integration point; however, the fully integrated solid element gives more reliable results than the element with just one integration point (LS-DYNA 2010). For the concrete slab, the concrete strength was 4000 psi; elastic modulus was 3.605e+006 psi and Poisson’s ratio was 0.3. The “Mat_Plastic_Kinematic” material model (elastic modulus = 2.900e+007 psi, tangent modulus 2.900e+006 psi, yield stress = 60ksi and Poisson’s ratio = 0.3) was used in conjunction with the 1-D beam element to model the rebars (LS-DYNA 2010). The actual rebar sizes were determined from the SCDOT drawings. In the finite element models, the 1-D beam elements (rebars) and the 3D 8-node solid elements (concrete) shared the same nodes (i.e. assumed not slip between the rebars and concrete).

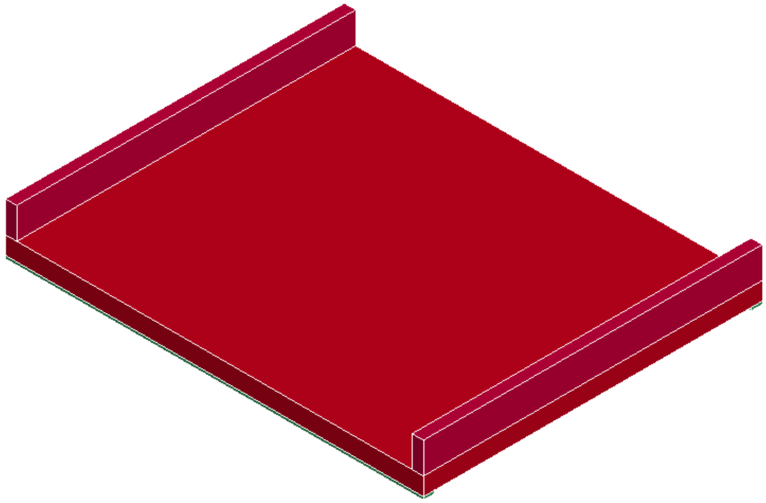


Figure 6.1: 3-D View of Archetype 1 Bridge Model.

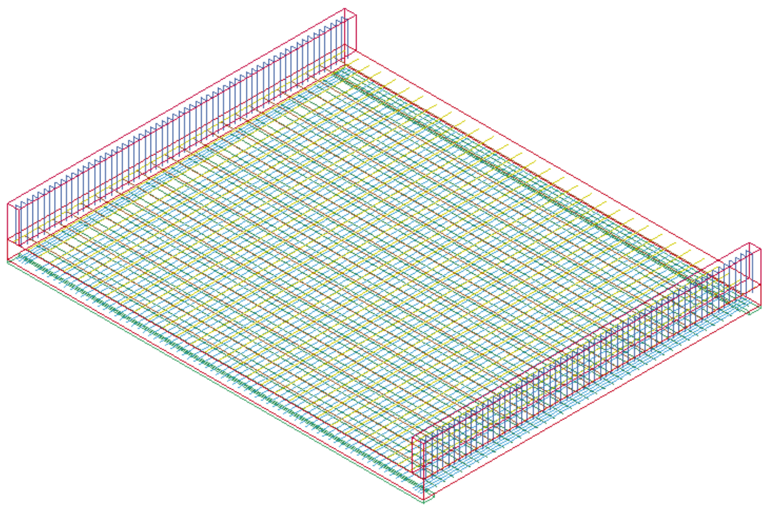


Figure 6.2: 3-D View of Rebars in the Archetype 1 Bridge Model.

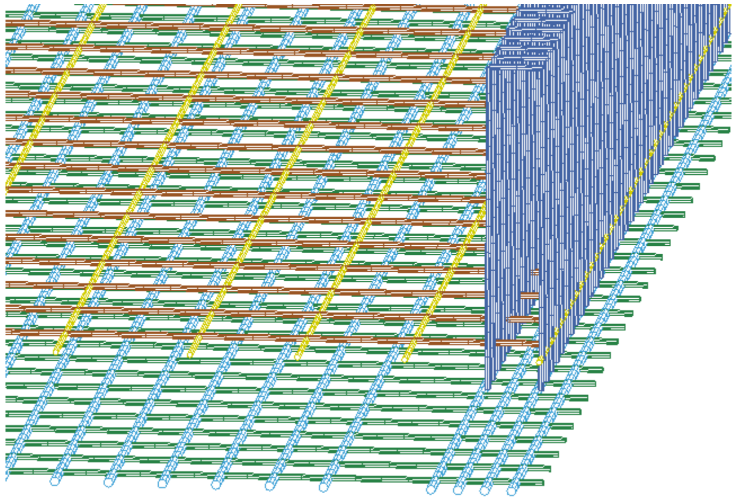


Figure 6.3: Zoom-In View of Rebars in the Archetype 1 Bridge Model.

Archetype Bridge 2, 3 and 4

Similar to the Archetype 1 bridge, the concrete slab for Archetypes 2, 3 and 4 bridges was modeled using the fully integrated 3-D 8-node solid elements. The actual bridge dimensions and girder sizes for each Archetype bridge were determined from their respective structural drawings. Both rebar and prestressing strands were modeled using the 1-D beam element. For the rebar element, the "Mat_Plastic_Kinematic" material model with the same material properties as the Archetype 1 bridge was utilized (LS-DYNA 2010). For the prestressing strands, the "Mat_Cable_Discrete_Beam" material model (elastic modulus = 2.900e+007 psi) was utilized to introduce prestressing force into the strands elements (LS-DYNA 2010). This material model does not allow compression forces to develop in the strands elements (LS-DYNA 2010).

Figure 6.4 to Figure 6.9 show the 3-D views and cross-sectional views of the Archetypes 2, 3 and 4 models. Note that while diaphragms are not shown in the cross-sectional views (Figures 6.5, 6.7, and 6.9), diaphragms were included in the FE models of Archetypes 2, 3 and 4 bridges.

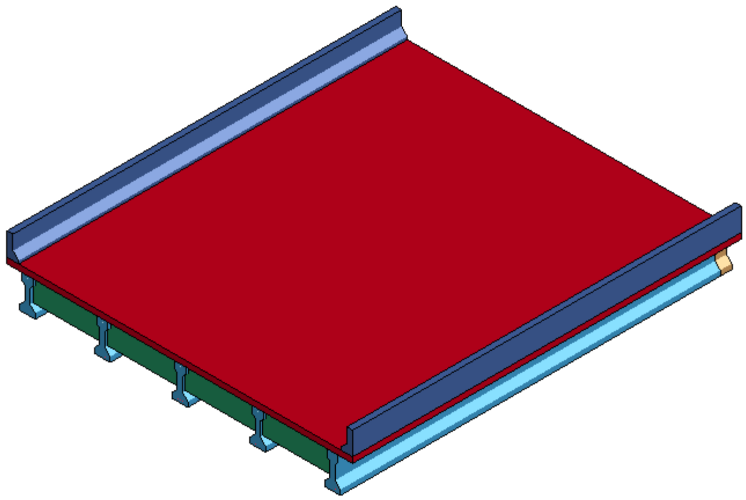


Figure 6.4: 3-D View of Archetype 2 Bridge Model.

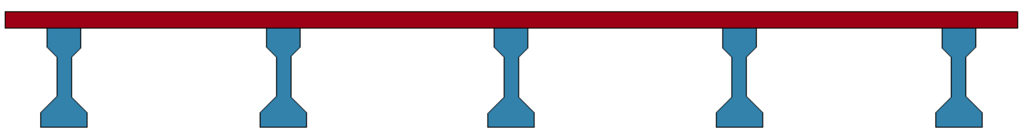


Figure 6.5: Cross-Sectional View of Archetype 2 Bridge Model.

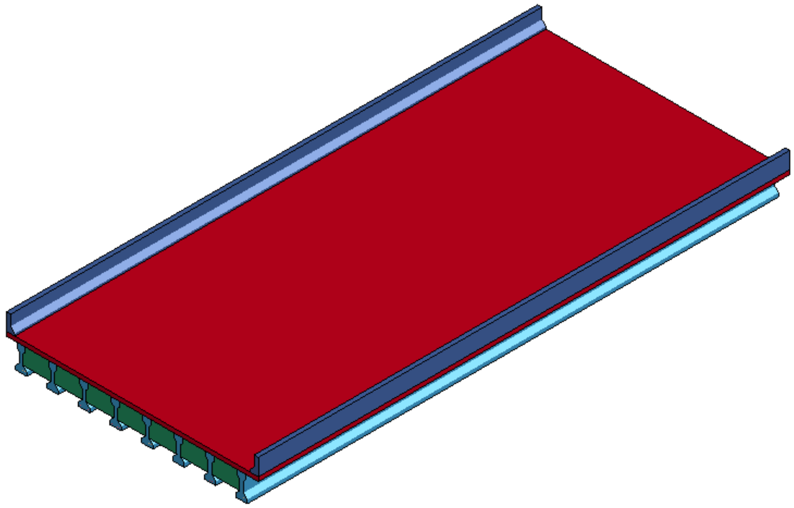


Figure 6.6: 3-D View of Archetype 3 Bridge Model.

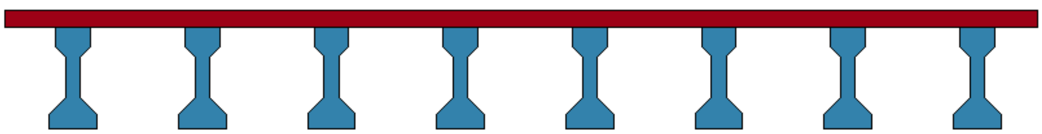


Figure 6.7: Cross-Sectional View of Archetype 3 Bridge Model.

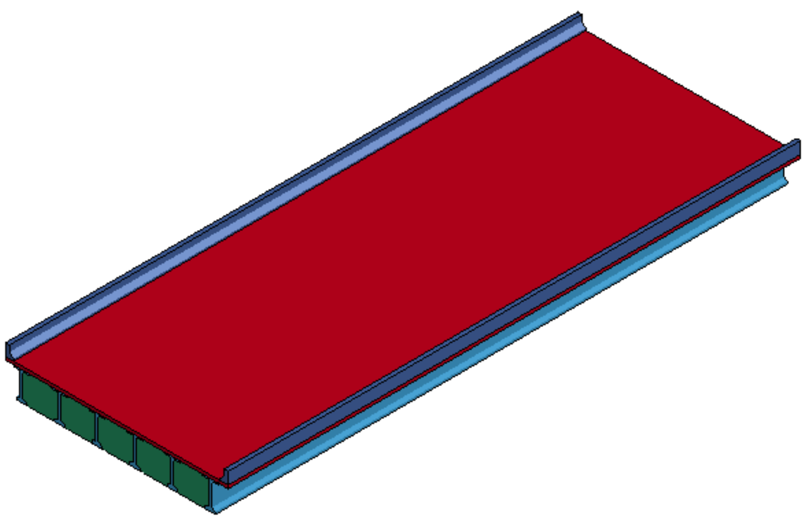


Figure 6.8: 3-D View of Archetype 4 Bridge Model.

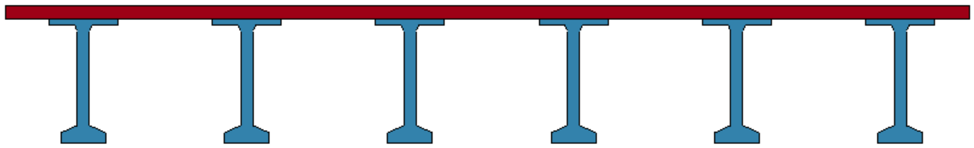


Figure 6.9: Cross-Sectional View of Archetype 4 Bridge Model.

Similar to the slab model, FE meshes for the girders of the Archetypes 2, 3 and 4 models were constructed using the 3-D solid and 1-D beam elements to represent the concrete and prestressing strands, respectively. Using a mesh with smaller elements generally produces better results but it also needs more computation time (LS-DYNA 2010). In order to keep the mesh size and the computation time at a reasonable level, it was deemed not feasible to model each prestressing strand in the girder as a separate element. In this study, several prestressing strands were lumped together in girder meshes.

Figure 6.10 (left) shows the actual strands arrangement at the mid span of the Archetype 2 girder. As can be seen, there were 2 top strands and 12 bottom strands in the girder (Barrett 2011). The corresponding FE mesh for the girder is shown in Figure 6.10 (right) where one line of strand elements in the top of the girder and five lines of strand elements in the bottom of the girder were utilized to represent the actual distribution of the strands. In the Archetype 2 FE model, one top strand element represented 2 prestressing strands while one bottom strand element represented 2.4 prestressing strands. Figure 6.11 shows the cross-sectional and isometric views of the LS-DYNA model for the girders of Archetype 2 bridge.

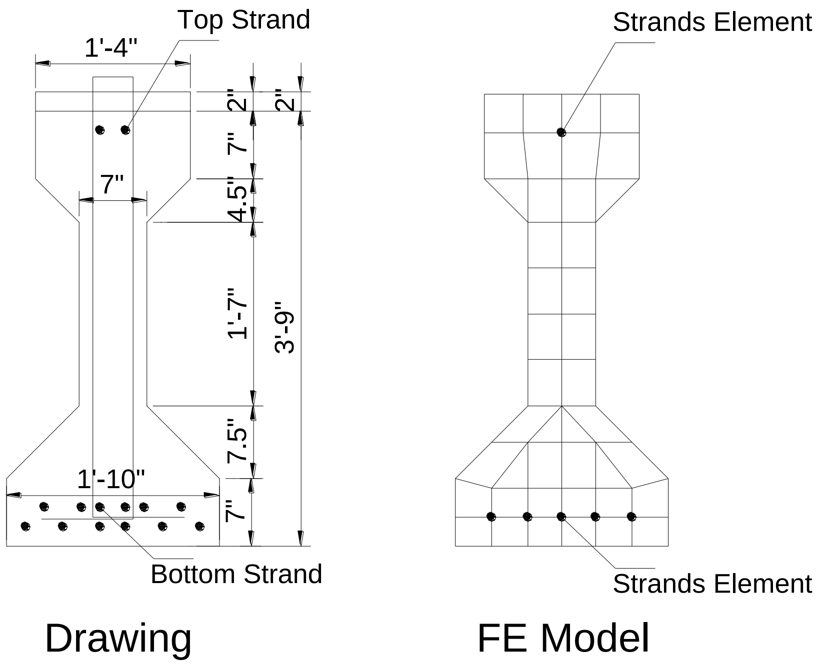


Figure 6.10: Cross-Sectional View of Archetype 2 Bridge Girder at Mid-Span: (Left) Actual Strands Distribution and (Right) Strand Elements in FE Model.

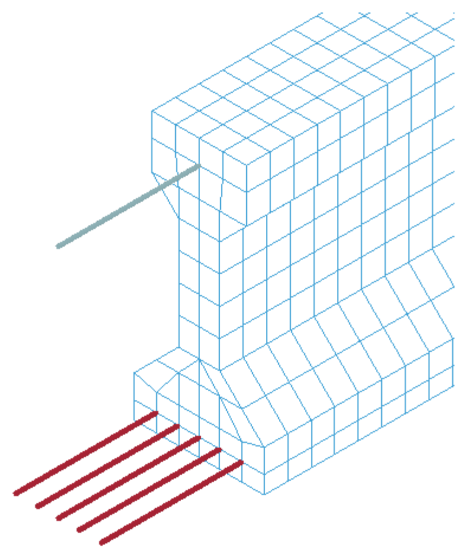


Figure 6.11: Zoom-In View of Strands at the Mid-Span of The Girder of Archetype 2 Bridge.

The same modeling technique was utilized in the FE models for the Archetypes 3 and 4 bridges. Figure 6.12 (left) and (right) shows the actual strands arrangement and the FE model strand layout at the mid span of the Archetype 3 bridge girder, respectively. The actual girder had 2 top strands and 30 bottom strands while in the FE model, one and ten lines of strand elements were utilized in the top and bottom of the girder, respectively. Figure 6.13 shows the cross-sectional and isometric views of strands in the LS-DYNA model for the girders of Archetype 3 Bridge.

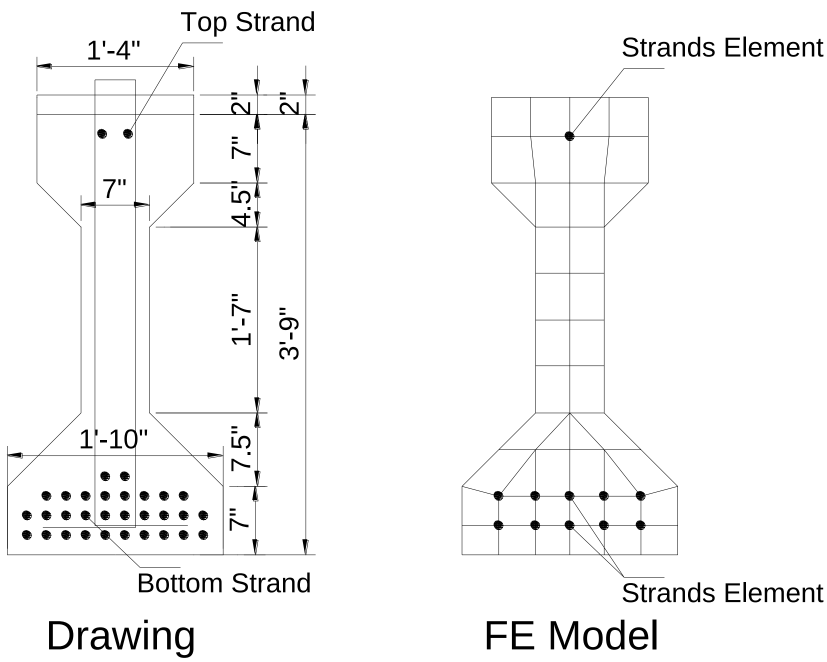


Figure 6.12: Cross-Sectional View of Archetype 3 Bridge Girder at Mid-Span: (Left) Actual Strands Distribution and (Right) Strand Elements in FE Model.

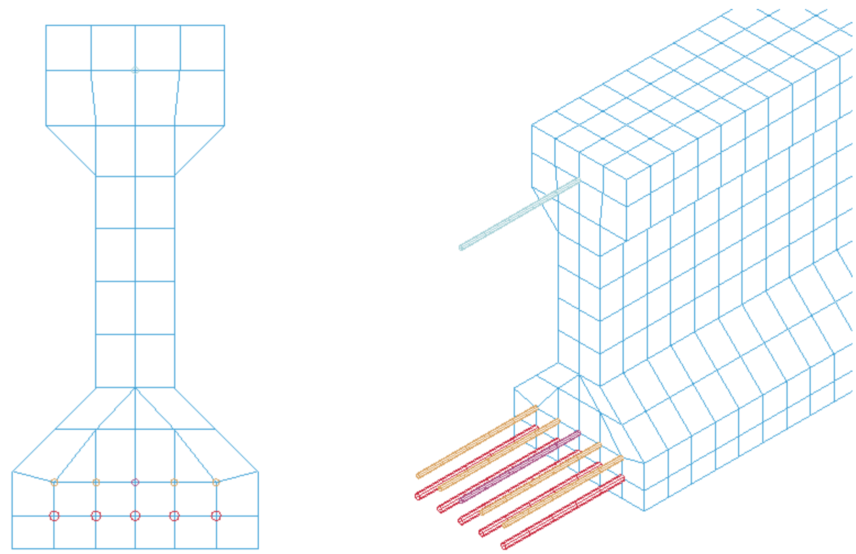


Figure 6.13: Zoom-In View of Strands at the Mid-Span of The Girder of Archetype 3 Bridge.

The cross-sectional views at the mid-span of Archetype 4 bridge girder were obtained from the actual structural drawings (Figure 6.14). As can be seen, there are 4 top strands and 42 bottom strands. For modeling purpose, the four top strands were lumped into one line of strand element and the 42 bottom strands were modeled using 12 lines of strand elements (see Figure 6.14). So for Archetype 4 bridge model, one top strand element represented four prestressing strands and one bottom strand element represented 1.6 to 6 prestressing strands, depending on its location. Figure 6.15 shows the LS-DYNA FE mesh for the girder and strands for Archetype 4 bridge.

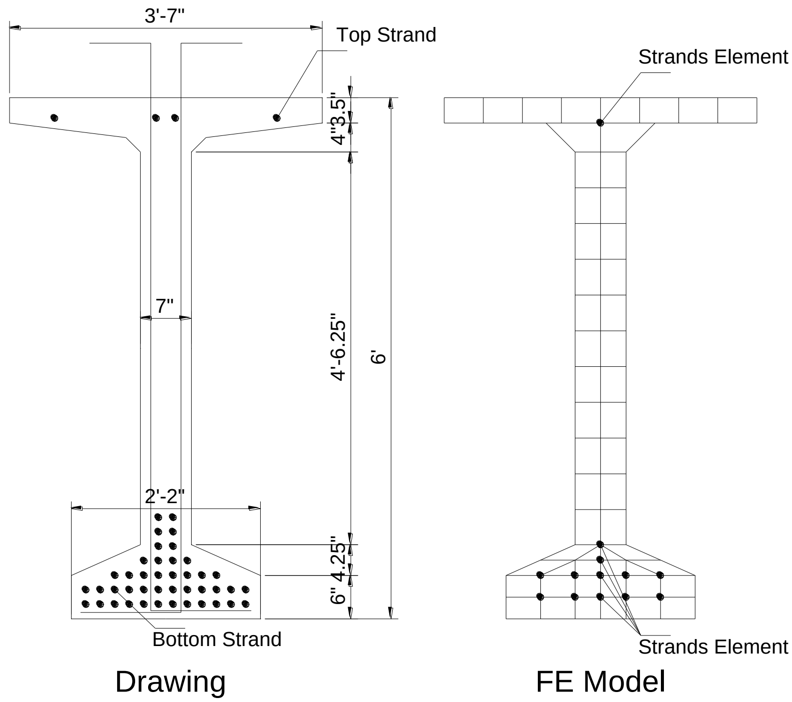


Figure 6.14: Cross-Sectional View of Archetype 4 Bridge Girder at Mid-Span: (Left) Actual Strands Distribution and (Right) Strand Elements in FE Model.

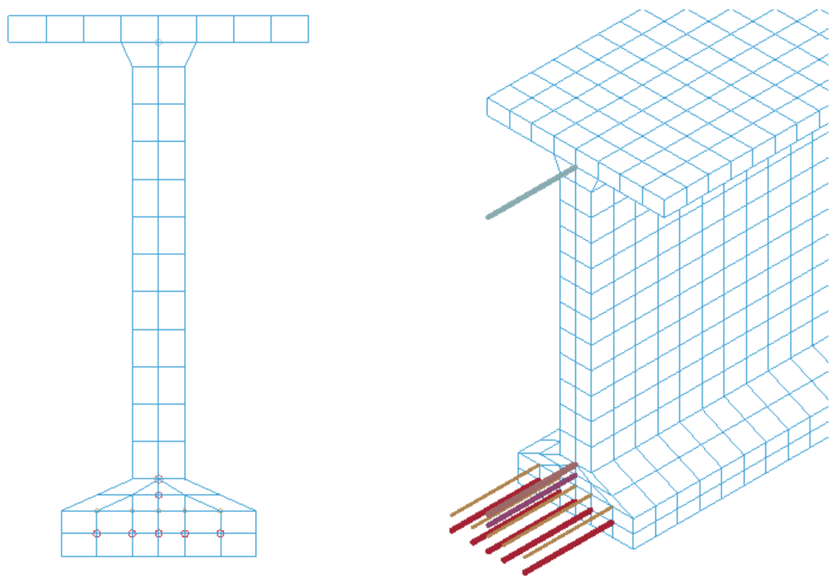


Figure 6.15: Zoom-In View of Strands at the Mid-Span of The Girder of Archetype 4 Bridge.

Since a strand element in the FE model represented more than one actual strand, the FE strands parameters including the prestressing forces and strand areas were adjusted accordingly based on the actual number of strands. Equivalent prestressing forces and strand areas were used for these strand elements. For example, for a strand element that represented two actual strands, its strand area was doubled in the FE model.

Tires in Finite Element Model

In order to realistically capture the dynamic interaction between the bridge and the moving truck, the air-bag function (LS-DYNA 2010) was utilized to model the truck tires and to distribute the truck weight to the bridge deck. To consider the dynamic effects, the tires were moved across the bridge at a prescribed travel speed, which was set to 60 miles per hour in this research. Figure 6.16 shows a tire before and after inflation. The "SURFACE_TO_SURFACE" contact analysis was applied between the tires and the bridge deck (LS-DYNA 2010). An elastic material with an elastic modulus of 1.381e+004 psi and a Poisson's ratio of 0.45 was used for the tire elements.

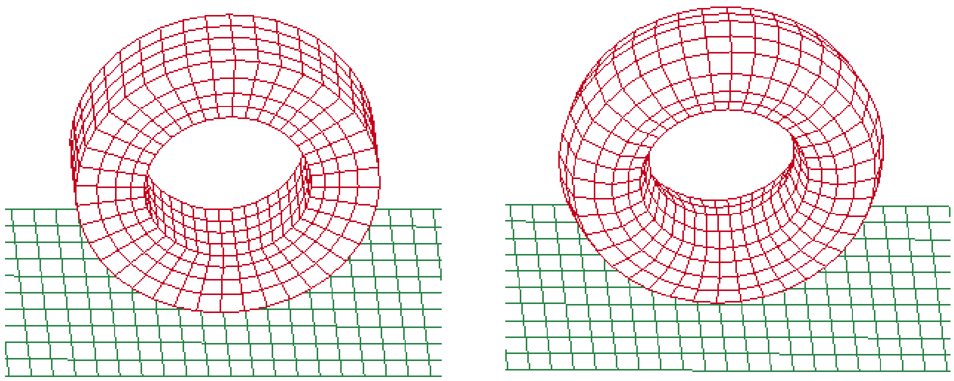


Figure 6.16: Tire Before and After Air Inflation.

Finite Element Model Results

For each of the Archetype bridge models, individual truck model[1] (see Table 4.7 and Table 4.8) was utilized to apply loading to the bridge and the maximum stress range experienced by the prestressing strands or steel rebar at the mid span was recorded for each truck model. For Archetype 1 bridge, the stress ranges of all longitudinal reinforcement rebars at the mid span were recorded and the maximum value was selected as the stress range for the fatigue analysis (discussed later in Chapter 8). Similarly, for Archetype 2, 3 and 4 bridges, the stress ranges of the bottom prestressing strands at the mid span were recorded and the maximum values were selected as the stress range for the

[1] A truck model is defined by three parameters, number of axles ,gross vehicle weight and axle spacing. See Table 4.7 and Table 4.8.

fatigue analysis. Figure 6.17 shows a typical element strain time-history output from LS-DYNA analysis. In Figure 6.17, the maximum strain and minimum strain during the analysis were recorded and the stress range was determined as the strain range multiplied by the elastic modulus of the strand.

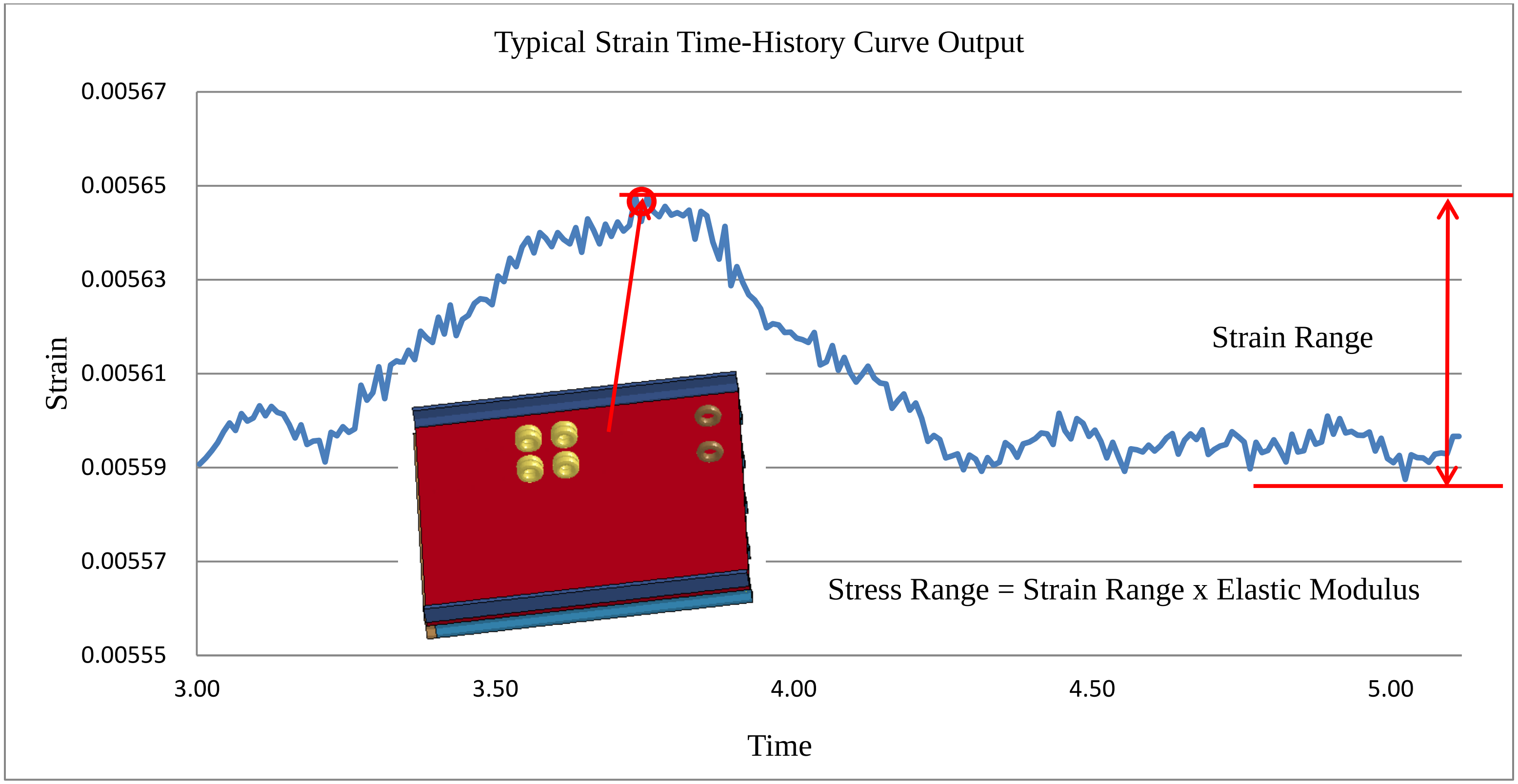


Figure 6.17: Typical Strain Time-History Results Curve.

Table 6.2 to Table 6.5 present the maximum stress ranges induced by each truck model for Archetype 1, 2, 3 and 4 bridges, respectively.

Table 6.2: Stress Range of Archetype 1 Bridge.

Axle Group	Truck Type	Stress Range of GVW1 (ksi)	Stress Range of GVW2 (ksi)	Stress Range of GVW3 (ksi)
2-Axle	A21	0.453	1.147	1.494
3-Axle	A31	N/A	N/A	1.338
	A32	0.633	0.755	N/A
4-Axle	A41	0.667	0.688	1.665
	A42	N/A	N/A	2.015
	A43	0.710	0.818	N/A
	A44	N/A	N/A	1.572
	A45	0.518	0.755	N/A
5-Axle	A51	N/A	N/A	2.099
	A52	0.744	0.913	N/A
6-Axle	A61	N/A	N/A	1.575
	A62	0.841	1.122	N/A
7-Axle	A71	N/A	N/A	2.561
	A72	0.736	1.220	N/A
8-Axle	A81	N/A	N/A	2.287
	A82	0.992	1.229	N/A

Table 6.3: Stress Range of Archetype 2 Bridge.

Axle Group	Truck Type	Stress Range of GVW1 (ksi)	Stress Range of GVW2 (ksi)	Stress Range of GVW3 (ksi)
2-Axle	A21	0.718	1.044	1.835
3-Axle	A31	N/A	N/A	2.082
	A32	1.163	1.650	N/A
4-Axle	A41	1.143	1.462	2.327
	A42	N/A	N/A	2.824
	A43	1.098	1.516	N/A
	A44	N/A	N/A	3.235
	A45	1.123	1.466	N/A
5-Axle	A51	N/A	N/A	3.206
	A52	1.098	1.579	N/A
6-Axle	A61	N/A	N/A	2.697
	A62	1.314	2.156	N/A
7-Axle	A71	N/A	N/A	4.244
	A72	1.439	2.762	N/A
8-Axle	A81	N/A	N/A	3.206
	A82	1.383	2.636	N/A

Table 6.4: Stress Range of Archetype 3 Bridge.

Axle Group	Truck Type	Stress Range of GVW1 (ksi)	Stress Range of GVW2 (ksi)	Stress Range of GVW3 (ksi)
2-Axle	A21	1.051	1.379	1.715
3-Axle	A31	N/A	N/A	2.129
	A32	1.421	1.939	N/A
4-Axle	A41	1.356	1.811	2.613
	A42	N/A	N/A	3.241
	A43	1.577	2.116	N/A
	A44	N/A	N/A	2.671
	A45	1.291	1.661	N/A
5-Axle	A51	N/A	N/A	3.136
	A52	1.540	1.968	N/A
6-Axle	A61	N/A	N/A	3.534
	A62	1.472	2.204	N/A
7-Axle	A71	N/A	N/A	5.802
	A72	1.607	2.684	N/A
8-Axle	A81	N/A	N/A	3.723
	A82	1.530	2.934	N/A

Table 6.5: Stress Range of Archetype 4 Bridge.

Axle Group	Truck Type	Stress Range of GVW1 (ksi)	Stress Range of GVW2 (ksi)	Stress Range of GVW3 (ksi)
2-Axle	A21	1.063	1.571	1.808
3-Axle	A31	N/A	N/A	2.378
	A32	1.493	1.904	N/A
4-Axle	A41	1.394	1.816	2.589
	A42	N/A	N/A	3.346
	A43	1.586	2.154	N/A
	A44	N/A	N/A	2.776
	A45	1.354	1.864	N/A
5-Axle	A51	N/A	N/A	2.842
	A52	1.733	2.282	N/A
6-Axle	A61	N/A	N/A	3.918
	A62	1.790	2.773	N/A
7-Axle	A71	N/A	N/A	5.614
	A72	1.998	3.143	N/A
8-Axle	A81	N/A	N/A	4.516
	A82	1.848	3.067	N/A

CHAPTER SEVEN

BRIDGE REPLACEMENT COST MODEL

Bridge Replacement Cost Models

In order to estimate the damage costs caused by truck traffic on bridges, the replacement costs of individual bridges must first be determined. The bridge replacement costs used in this study were derived from the bridge replacement cost database in the HAZUS-MH program (HAZUS 2003). It should be noted that the HAZUS-MH is developed for loss estimation under extreme natural hazard events (e.g. earthquakes); hence not all the bridges are accounted for in the HAZUS-MH program. The HAZUS-MH database contains the replacement costs for a proximately half of the bridges in South Carolina (4,096 bridges). The total number of bridges in South Carolina is 9,271. For those bridges that are not in the HAZUS-MH database, their replacement costs were estimated using the *bridge cost models*, developed as part of this study using the replacement costs of the 4,096 bridges available in the HAZUS-MH database.

The first step in developing the bridge cost model was to match the longitude and latitude coordinates of the 4,096 bridges with known replacement costs in the HAZUS program to that in the NBI database. Next, the 9,271 bridges in NBI database were grouped together according to their material type and structural type (Table 7.1).

Table 7.1: Bridge Cost Group.

Cost Model Number	Material Type	Structure Type
1	Concrete	Slab
2	Concrete	Stringer/Multi-Beam or Girder
3	Concrete	Girder and Floor Beam System
4	Concrete	Tee Beam
5	Concrete	Box Beam or Girders - Multiple
6	Concrete	Frame (except frame culverts)
7	Concrete	Arch - Deck
8	Concrete	Tunnel
9	Concrete	Culvert (includes frame culverts)
10	Concrete	Channel Beam
11	Concrete	Other
12	Concrete Continuous	Slab
13	Concrete Continuous	Stringer/Multi-Beam or Girder
14	Concrete Continuous	Tee Beam
15	Concrete Continuous	Box Beam or Girders - Multiple
16	Concrete Continuous	Box Beam or Girders - Single or Spread
17	Steel	Slab
18	Steel	Stringer/Multi-Beam or Girder
19	Steel	Girder and Floor Beam System

Table 7.1 (continued): Bridge Cost Group.

Cost Model Number	Material Type	Structure Type
20	Steel	Frame (except frame culverts)
21	Steel	Truss - Thru
22	Steel	Arch - Deck
23	Steel	Movable - Bascule
24	Steel	Movable - Swing
25	Steel	Culvert (includes frame culverts)
26	Steel	Other
27	Steel Continuous	Slab
28	Steel Continuous	Stringer/Multi-Beam or Girder
29	Steel Continuous	Girder and Floor Beam System
30	Steel Continuous	Frame (except frame culverts)
31	Steel Continuous	Truss - Thru
32	Steel Continuous	Stayed Girder
33	Steel Continuous	Movable - Swing
34	Prestressed Concrete	Slab
35	Prestressed Concrete	Stringer/Multi-Beam or Girder
36	Prestressed Concrete	Girder and Floor Beam System
37	Prestressed Concrete	Tee Beam
38	Prestressed Concrete	Box Beam or Girders - Multiple
39	Prestressed Concrete	Channel Beam
40	Prestressed Concrete	Other

Table 7.1 (continued): Bridge Cost Group.

Cost Model Number	Material Type	Structure Type
41	Prestressed Concrete Continuous	Slab
42	Prestressed Concrete Continuous	Stringer/Multi-Beam or Girder
43	Prestressed Concrete Continuous	Segmental Box Girder
44	Wood or Timber	Slab
45	Wood or Timber	Stringer/Multi-Beam or Girder
46	Masonry	Arch - Deck
47	Masonry	Culvert (includes frame culverts)
48	Aluminum, Wrought Iron, or Cast Iron	Culvert (includes frame culverts)
49	Other	Slab
50	Other	Other

For those bridge cost groups that have more than five known bridge replacement costs (obtained from the HAZUS-MH database), the bridge replacement costs were fitted to two power equations, one as a function of the total structure length (Equation 7.1) , and the other as a function of the total structure area (Equation 7.2).

$$C_{R1} = a_1 L^{b_1} \tag{7.1}$$

where

C_{R1} is the bridge replacement cost as a function of the total structure length

L is the total structure length

a_1 and b_1 are fitted distribution parameters for Equation (7.1)

$$C_{R2} = a_2 A^{b_2} \tag{7.2}$$

where

C_{R2} is the bridge replacement cost as a function of the total structure area

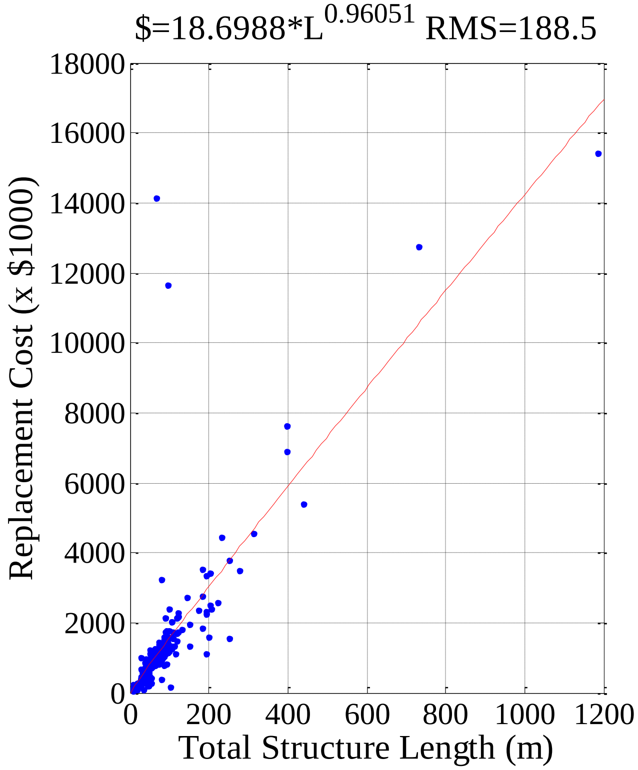
A is the total structure area

a_2 and b_2 are fitted distribution parameters for Equation (7.2)

Figure 7.1 and Figure 7.2 give two example replacement cost models for the prestressed concrete girder. The data points shown in Figures 7.1 and 7.2 represent the known bridge replacement cost values obtained from the HAZUS-MH database. For each bridge cost group, the RMS (root mean square) errors of the fitted power equation curves for both the total structure length and total area models (i.e. Equations 7.1 and 7.2) were calculated. The model with the smaller RMS value was selected as the cost model for the bridge cost group. The selected model or equation was then used to compute the replacement costs of those bridges that were not accounted for in the HAZUS-MH database.

For the bridge cost groups that have less than five known bridge replacement costs, an average unit area cost was determined and used as the replacement cost to compute the replacement costs for the rest of the bridges in the same cost group. For bridge cost groups that were unable to establish a cost model or unit area cost, a cost model or unit area cost from a similar bridge cost group was assigned to this cost group. The complete details for the cost models and the fitted cost model parameters can be found in Appendix C.

5 Prestressed concrete *; 02 Stringer/Multi-beam or Girder N=381(1286) Mean Unit Cost: 1.4964x $1000/m^2



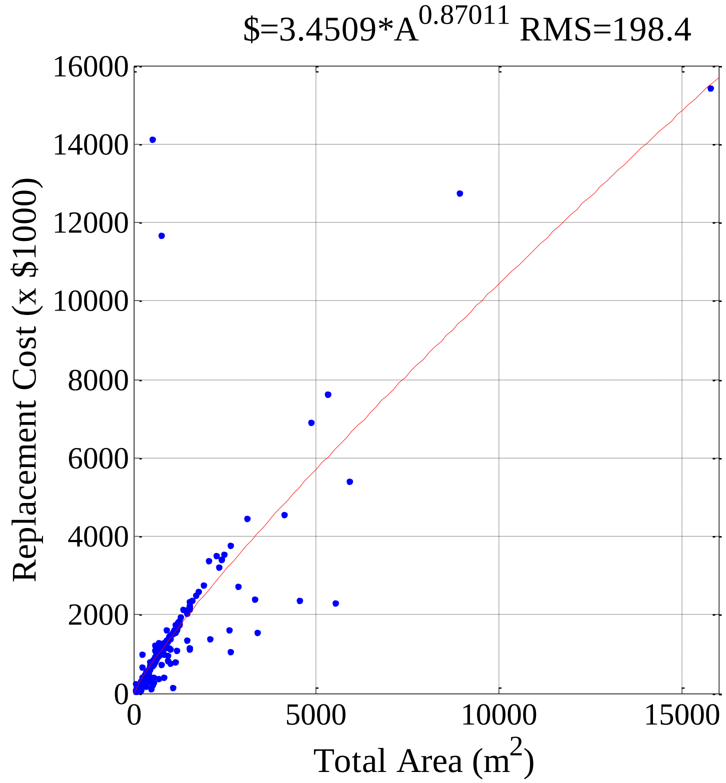


Figure 7.1: Replacement Cost Model for Cost Model 35.

Figure 7.1 shows the replacement cost model for multi-girder prestressed concrete bridges. The data points are the known replacement costs from the HAZUS-MH program and the red curves are the least-squares fits of the replacement costs using Equations 7.1 and 7.2. The left figure is the replacement cost model expressed as a function of the total structural length and the right figure is the replacement cost model expressed in terms of the total bridge area. The fitted equations for both models are also shown in the figure. As can be seen from the figure, the model with the total length as the predictor had a smaller RMS (188.5) than the model using the total area as the predictor (198.4); therefore, the total structure length model was selected to estimate the replacement cost for all bridges in this bridge cost group.

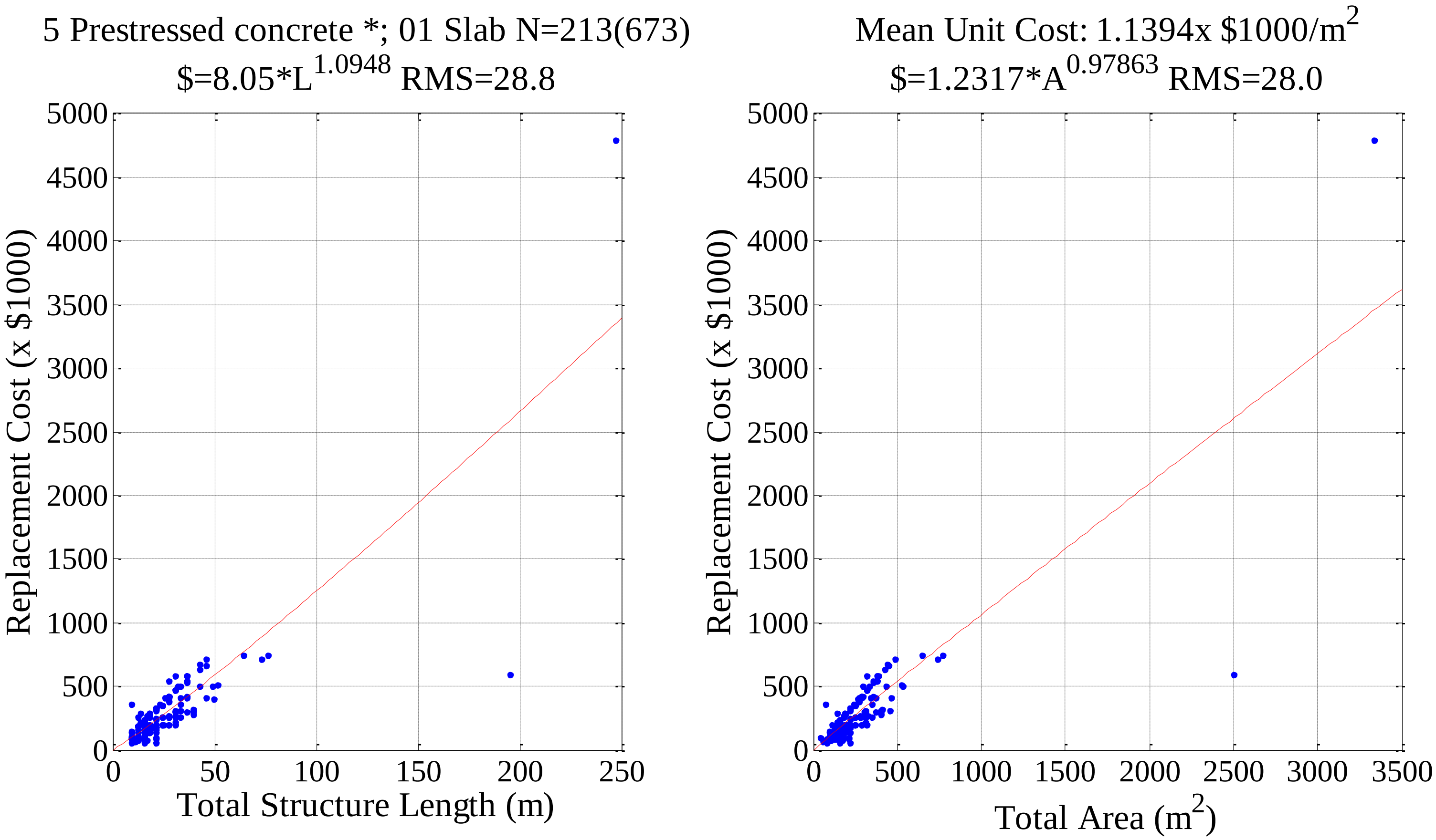


Figure 7.2: Replacement Cost Model for Cost Model 34.

Figure 7.2 shows the two candidate replacement cost models prestressed concrete slab bridges. For prestressed concrete slab bridges, the fitted cost model using the total length had a larger RMS (28.8) than that of the total area model (28); In this case, the cost model with the total structure area as the predictor was utilized to estimate the replacement costs of the remaining prestressed slab bridges that were without cost information.

Once the bridge cost models for different bridge types were developed, the replacement cost for each bridge in the NBI database was able to be determined. The histogram in Figure 7.3 shows the distribution of bridge replacement costs in South Carolina. The replacement costs for the majority of the bridges are less than $3 million dollars (2003 US Dollar). Figure 7.4 shows the geographical distribution of the bridge replacement costs. As expected, the majority of bridges with replacement cost of greater than $1 million dollars (2003 US Dollar) are along the main highway routes. These bridge replacement costs were used in conjunction with the fatigue analysis results to determine the annual damage costs for individual bridges.

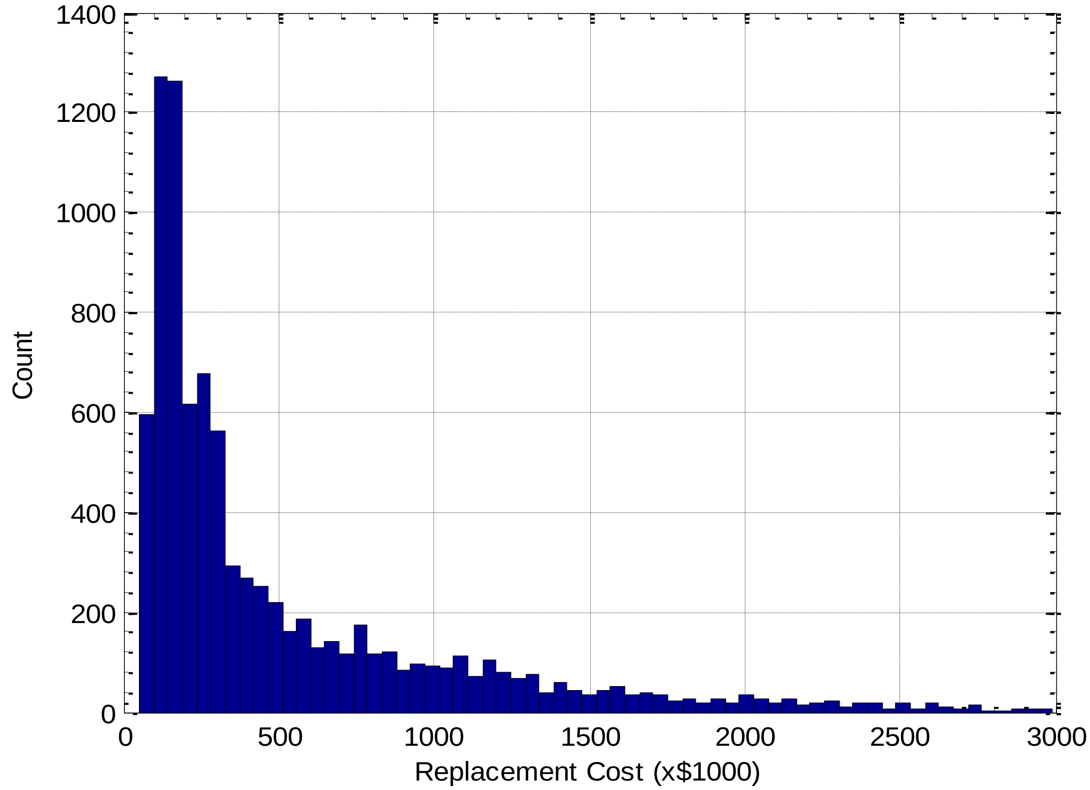


Figure 7.3: Distribution of South Carolina Bridge Replacement Costs.

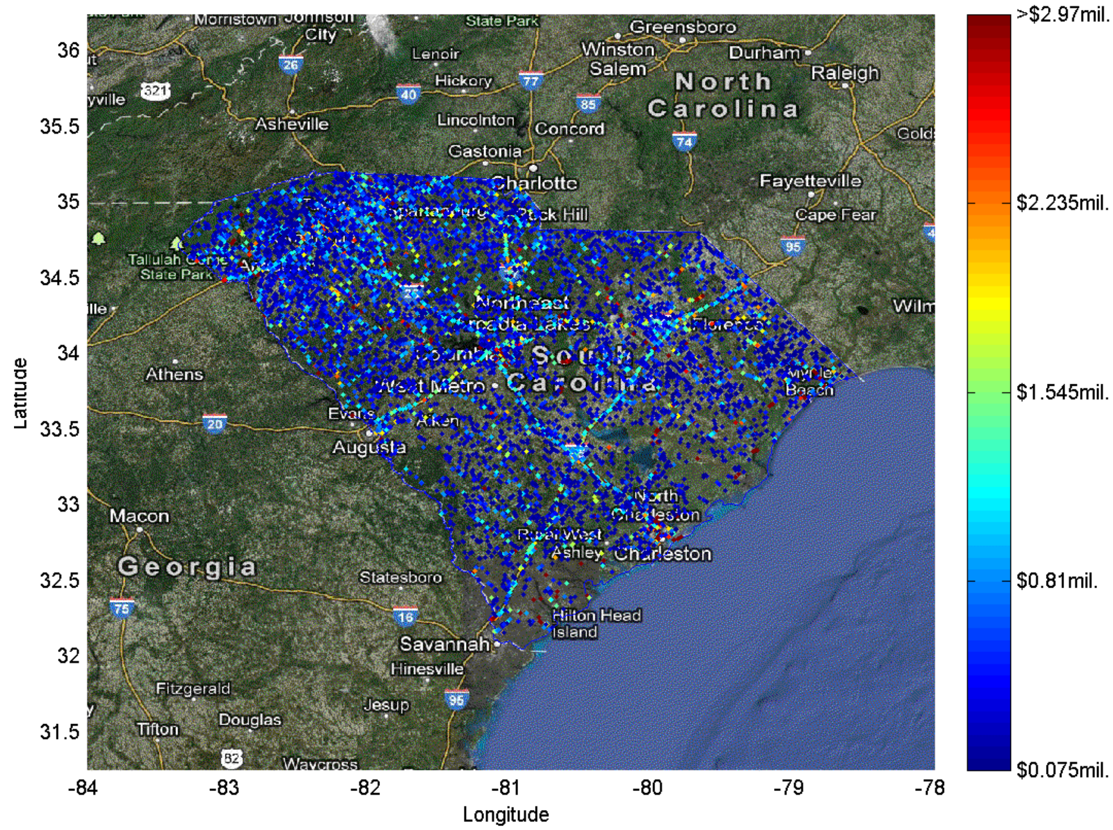


Figure 7.4: Geographical Distribution of South Carolina Bridge Replacement Costs.

The total replacement cost for all bridges in South Carolina was determined to be approximately $7.615 billion dollars (2003 US Dollar). Note that the estimated total bridge asset value was derived from the bridge replacement cost database in the HAZUS-MH program, which was based on the 2003 US dollar. The average consumer price index (CPI) from 2004 to 2011 was used to convert the bridge cost to 2011 US dollar. The year of 2011 was selected because the average daily truck traffic used in the fatigue damage analysis was based on the 2011 data. By substituting the average CPI from 2004 to 2011, 2.575% (Table 7.2) (Bureau of Labor Statistics 2013), into Equation

7.3, the total bridge replacement cost in 2011 US dollar was found to be \$9.332 billion dollars.

$$\text{Total Replacement Cost} = 7.615 \times (1 + 2.575\%)^8 = 9.332\ Billion \qquad (7.3)$$

Table 7.2: Average CPI from 2004 to 2011.

Year	CPI (%)
2004	3.3
2005	3.4
2006	2.5
2007	4.1
2008	0.1
2009	2.7
2010	1.5
2011	3.0
Average	2.575

CHAPTER EIGHT

BRIDGE FATIGUE LIFE

Introduction

The bridge fatigue life is defined as the number of allowable stress cycles under a given stress range, referred herein as the N value. The N value can be computed using Equation (2.2) and Equation (2.3) for concrete slab (Archetype 1) and prestresstred concrete (Archetypes 2 to 4) bridges, respectively. It should be noted that Equations (2.2) and (2.3) are for the *strength-level fatigue limit state* (i.e. fatigue fracture of rebars or prestressing strands). The endurance limit for both the rebars and the prestressing strands is 20 ksi. Based on the FE analysis results (see Table 6.2 to Table 6.5), it can be seen that all stress ranges are less than the endurance limit, which indicates that the bridges have unlimited number of stress cycles (or infinite fatigue life). Per AASTHO design specification (AASHTO 2007), bridges are designed with a limited service life of 75 years. So while the strength-level limit state Equations (2.2) and (2.3) suggest that fatigue fracture of rebars or prestressing strands will not occur over the design lifetime (i.e. 75 years), it is not realistic to expect the bridges to have infinite *service life* under repetitive fatigue loading, in particular with heavy overweight trucks. A recent study (Bathias and Paris 2005) shows that under extreme large number of stress cycles (in Giga-Cycle range), the N value (i.e. fatigue life) will further decrease. Based on the study by Bathias and Paris (2005) and the target design life of 75 years, a *service-level fatigue limit state* is defined to estimate the bridge fatigue damage. This service-level fatigue limit state is derived from the strength-level fatigue limit state curve and calibrated using the target design bridge life (i.e. 75 years). The discussion of this service-level fatigue limit state is provided in following sections.

Service-Level Fatigue Limit State for Archetype 1 Bridge

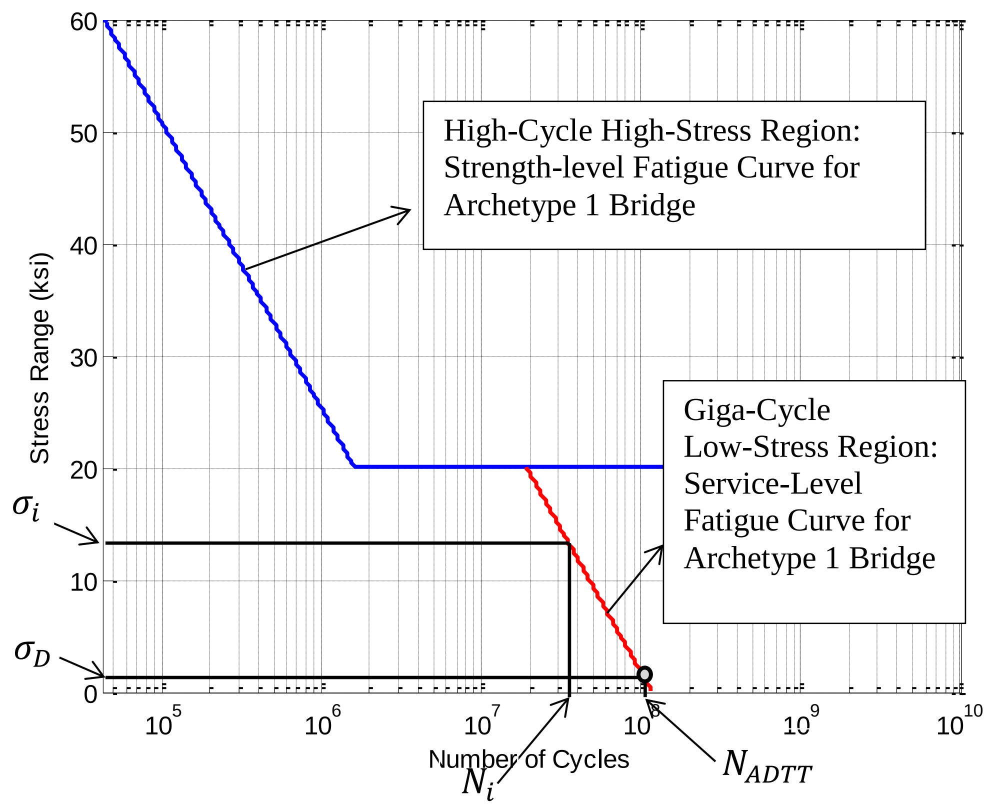


Figure 8.1: Strength-Level and Service-Level Fatigue Curves for Archetype 1 Bridge.

Figure 8.1 shows both the strength-level fatigue limit state curve and service-level fatigue limit state curve for Archetype 1 bridge. In this figure, the vertical axis represents the stress ranges and the horizontal axis represents the *N* number, which is the number of cycles the bridge can sustain for a given stress range. In this study, when the stress range was more than 20ksi (i.e. in the high-cycle high-stress region), the strength-level fatigue limit state (Equation (2.2)) is used to calculate the *N* number. In the Giga-Cycle region where stress range is less than 20ksi, a service-level fatigue limit state was derived and was used to calculate the N number.

The AASTHO fatigue design truck and target bridge life (i.e. 75 years) were used to derive the service-level fatigue limit state equation. Table 8.1 shows the stress ranges caused by the AASHTO fatigue design truck (σ_D) on the four Archetype bridges.

Table 8.1: LRFD Fatigue Design Truck Stress Range.

	Archetype 1	Archetype 2	Archetype 3	Archetype 4
LRFD Fatigue Design Truck Stress Range (ksi)	0.708	1.086	1.772	1.834

The corresponding *N* number (N_{ADTT}) for the stress ranges caused by the fatigue design truck (σ_D) can be calculated using the following equation:

$$N_{ADTT} = 4000 \times 365 \times 75 \quad (8.1)$$

where 4000 is the design average daily truck traffic (ADTT), which was determined from the AASHTO LRFD specification (AASHTO 2007) assuming the maximum average daily traffic (ADT) of 20,000 per lane and rural interstate truck traffic fraction of 0.2 (AASHTO 2007). The design ADTT computed using Equation (8.1) is given in Table 8.2.

Table 8.2: LRFD Fatigue Design Truck Allowable Number of Passing.

ADTT	Days	Years	LRFD Fatigue Design Truck Allowable Number of Passing
4000	365	75	1.10E+08

The stress ranges caused by the AASHTO fatigue truck and the design ADTT yield the Giga-cycle region (see Figure 8.1). According to Bathias and Paris (2005), the slope of the fatigue curve corresponds to the low-stress and extreme high-cycle region is similar to that of the high-stress region (i.e. Equation (2.2)). For convenience, Equation (2.2) is re-presented here:

$$\text{Log N} = 4.419 - 0.0392 \times \sigma - 0.013 \times \sigma_{min} + 0.0079 \times \text{G} + 7.8059 \times \text{D}_{nom} - 8.4155 \times \text{D}_{nom}{}^{2} + 2.799 \times \text{D}_{nom}{}^{3} \quad (8.2)$$

where

N: fatigue life in number of stress cycles for fatigue design truck, from Table 8.2

σ_{min}: minimum stress during stress cycle, (1.34ksi under self-weight)

G: rebar yield strength 60 ksi

D_{nom}: nominal rebar diameter 1.128 inch

σ: fatigue design truck stress range from Table 8.1

Substitute (N_{ADTT}) and the stress range of the design fatigue truck for Archetype 1 bridge into Equation (8.2) while keeping the slope of Equation 8.2 constant (i.e. 0.0392) yields the following equation for Archetype 1 bridge:

$$\text{Log N} = 8.0672 - 0.0392 \times \sigma \quad (8.3)$$

Equation (8.3), service-level fatigue limit state equation, was used to calculate the fatigue life and fatigue damage cost of Archetype 1 bridge.

The combined fatigue limit state curve (i.e. including the strength-level fatigue limit state and service-level fatigue limit state) is shown in Figure 8.1, where σ_D represents the fatigue design truck stress range and σ_i represents the stress range caused by an arbitrary truck model. N_{ADTT} is the number of expected cycles under the fatigue design truck (Table 8.2) while N_i is the allowable number of cycles under the stress range caused by an arbitrary truck (with a given axle configuration and weight).

Service-Level Fatigue Limit State for Archetype 2, 3 and 4 Bridges

The same concept and procedure used to determine the service-level fatigue limit state equation for Archetype 1 bridge were adopted and applied to Archetype 2, 3 and 4 bridges. The strength-level fatigue limit state Equation (2.3) for prestressing strands is repeated here for convenience.

$$\text{Log N} = 11 - 3.5 \times \text{Log}\,\sigma \tag{8.4}$$

where

N: fatigue life in number of stress cycles for fatigue design truck, from Table 8.2.

σ: fatigue design truck stress range from Table 8.1

Substitute N_{ADTT} and the stress ranges caused by the AASHTO fatigue design truck on Archetype 2, 3 and 4 bridges into Equation (8.4) while maintaining the slope of Equation (8.4) yields the following set of three service-level fatigue limit state equations for Archetype 2, 3 and 4 bridges, respectively:

Archetype 2 Bridge:

$$\text{Log N} = 8.1648 - 3.5 \times \text{Log}\sigma \tag{8.5}$$

Archetype 3 Bridge:

$$\text{Log N} = 8.909 - 3.5 \times \text{Log}\sigma \tag{8.6}$$

Archetype 4 Bridge:

$$\text{Log N} = 8.9613 - 3.5 \times \text{Log}\sigma \tag{8.7}$$

The strength-level and service-level fatigue limit state equations for all Archetype bridges are shown in Figure 8.2.

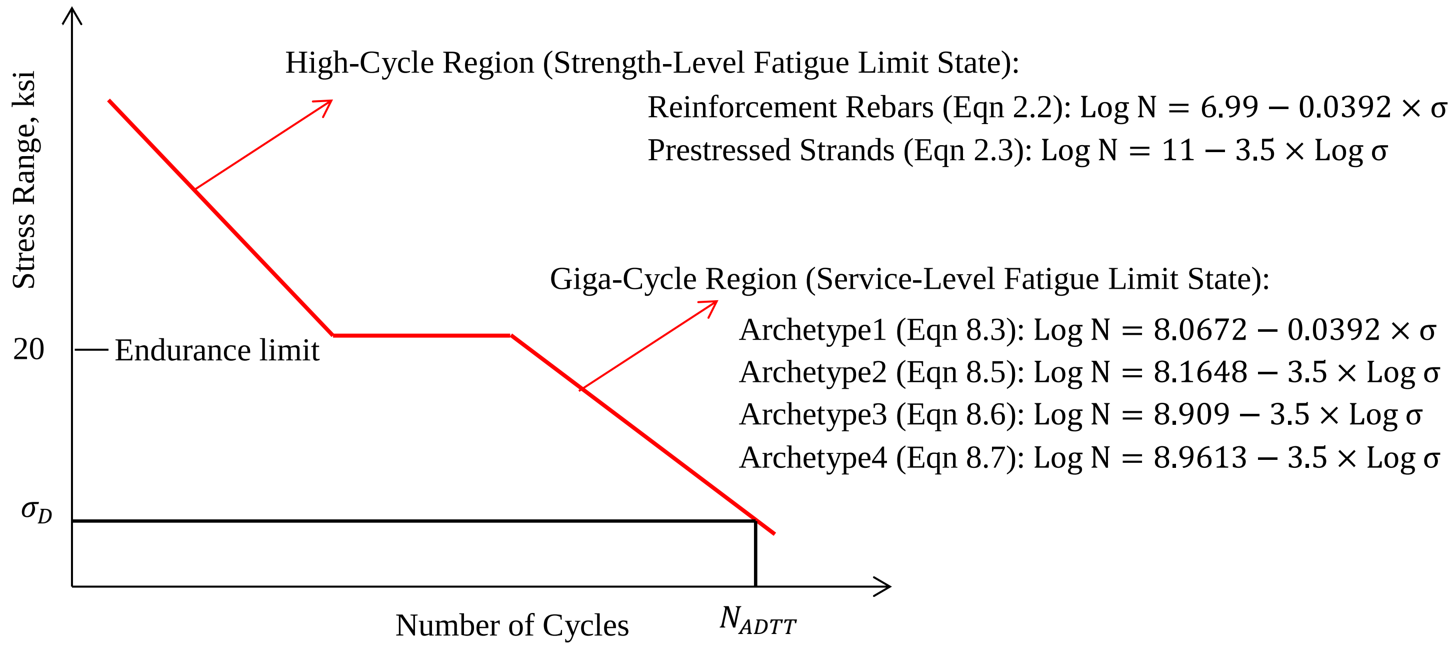


Figure 8.2 Strength-Level and Service-Level Fatigue Curves and Equations

The allowable bridge fatigue cycles (*N* values) for all truck models (different axle configurations and gross vehicle weights) and for all four Archetype bridges were calculated using the fatigue limit state equations presented in Figure 8.2. The results are presented in Table 8.3 to Table 8.6.

Table 8.3: Bridge Fatigue Life of Archetype 1 Bridge.

Axle Group	Truck Type	Allowable Number of Passing for GVW1	Allowable Number of Passing for GVW2	Allowable Number of Passing for GVW3
2-Axle	A21	1.12E+08	1.05E+08	1.02E+08
3-Axle	A31	N/A	N/A	1.03E+08
	A32	1.10E+08	1.09E+08	N/A
4-Axle	A41	1.10E+08	1.10E+08	1.00E+08
	A42	N/A	N/A	9.73E+07
	A43	1.09E+08	1.08E+08	N/A
	A44	N/A	N/A	1.01E+08
	A45	1.11E+08	1.09E+08	N/A
5-Axle	A51	N/A	N/A	9.66E+07
	A52	1.09E+08	1.07E+08	N/A
6-Axle	A61	N/A	N/A	1.01E+08
	A62	1.08E+08	1.05E+08	N/A
7-Axle	A71	N/A	N/A	9.26E+07
	A72	1.09E+08	1.05E+08	N/A
8-Axle	A81	N/A	N/A	9.50E+07
	A82	1.07E+08	1.04E+08	N/A

Table 8.4: Bridge Fatigue Life of Archetype 2 Bridge.

Axle Group	Truck Type	Allowable Number of Passing for GVW1	Allowable Number of Passing for GVW2	Allowable Number of Passing for GVW3
2-Axle	A21	4.66E+08	1.26E+08	1.75E+07
3-Axle	A31	N/A	N/A	1.12E+07
	A32	8.62E+07	2.53E+07	N/A
4-Axle	A41	9.15E+07	3.87E+07	7.60E+06
	A42	N/A	N/A	3.86E+06
	A43	1.05E+08	3.41E+07	N/A
	A44	N/A	N/A	2.40E+06
	A45	9.74E+07	3.83E+07	N/A
5-Axle	A51	N/A	N/A	2.48E+06
	A52	1.05E+08	2.95E+07	N/A
6-Axle	A61	N/A	N/A	4.54E+06
	A62	5.62E+07	9.93E+06	N/A
7-Axle	A71	N/A	N/A	9.28E+05
	A72	4.09E+07	4.17E+06	N/A
8-Axle	A81	N/A	N/A	2.48E+06
	A82	4.70E+07	4.91E+06	N/A

Table 8.5: Bridge Fatigue Life of Archetype 3 Bridge.

Axle Group	Truck Type	Allowable Number of Passing for GVW1	Allowable Number of Passing for GVW2	Allowable Number of Passing for GVW3
2-Axle	A21	6.81E+08	2.63E+08	1.23E+08
3-Axle	A31	N/A	N/A	5.76E+07
	A32	2.37E+08	7.99E+07	N/A
4-Axle	A41	2.79E+08	1.01E+08	2.81E+07
	A42	N/A	N/A	1.32E+07
	A43	1.65E+08	5.88E+07	N/A
	A44	N/A	N/A	2.60E+07
	A45	3.32E+08	1.37E+08	N/A
5-Axle	A51	N/A	N/A	1.48E+07
	A52	1.79E+08	7.58E+07	N/A
6-Axle	A61	N/A	N/A	9.77E+06
	A62	2.10E+08	5.10E+07	N/A
7-Axle	A71	N/A	N/A	1.72E+06
	A72	1.54E+08	2.56E+07	N/A
8-Axle	A81	N/A	N/A	8.15E+06
	A82	1.83E+08	1.87E+07	N/A

Table 8.6: Bridge Fatigue Life of Archetype 4 Bridge.

Axle Group	Truck Type	Allowable Number of Passing for GVW1	Allowable Number of Passing for GVW2	Allowable Number of Passing for GVW3
2-Axle	A21	7.39E+08	1.88E+08	1.15E+08
3-Axle	A31	N/A	N/A	4.41E+07
	A32	2.25E+08	9.60E+07	N/A
4-Axle	A41	2.86E+08	1.13E+08	3.28E+07
	A42	N/A	N/A	1.33E+07
	A43	1.82E+08	6.24E+07	N/A
	A44	N/A	N/A	2.57E+07
	A45	3.17E+08	1.03E+08	N/A
5-Axle	A51	N/A	N/A	2.36E+07
	A52	1.34E+08	5.10E+07	N/A
6-Axle	A61	N/A	N/A	7.68E+06
	A62	1.19E+08	2.58E+07	N/A
7-Axle	A71	N/A	N/A	2.18E+06
	A72	8.11E+07	1.66E+07	N/A
8-Axle	A81	N/A	N/A	4.67E+06
	A82	1.07E+08	1.81E+07	N/A

CHAPTER NINE

ANNUAL BRIDGE COST

Introduction

The annual bridge cost considered in this study included two components: (1) the annual bridge fatigue damage cost due to truck traffic, and (2) routine bridge maintenance cost. The annual bridge maintenance cost was obtained directly from the SCDOT bridge maintenance division, while the bridge damage cost was obtained using the fatigue analysis. The procedure for determining the annual fatigue damage cost is summarized in the following steps:

- Step 1, Compute the allowable bridge fatigue life (N) for each truck model (i) using the FE analysis results (see Chapter 8)
- Step 2: Compute the annual consumed bridge fatigue life (N_C) for each truck model
- Step 3, Compute the annual bridge fatigue damage (D)
- Step 4, Determine the bridge replacement cost (C_R) (see Chapter 7)
- Step 5, Compute the annual bridge fatigue damage cost (C_D)

Once the total annual bridge fatigue damage cost in South Carolina was calculated using the steps shown above. The annual bridge maintenance cost was then added to the fatigue damage cost to obtain the total annual bridge cost in South Carolina. More details for each step are discussed in the following sections.

Annual Bridge Fatigue Damage Cost Sample Calculation

The annual bridge damage cost is bridge type and site specific (i.e. it depends on the truck traffic). For discussion purpose, a bridge site with daily average truck traffic (ADTT) of 4000 is assumed for the following sample calculations.

Step 1, Compute the allowable bridge fatigue life (N) *for each truck model using the FE analysis results (see Chapter 8)*

The allowable bridge fatigue life in terms of the number of passages allowed for each truck model (*N*) was computed using the methodology discussed in Chapter 8. The results for all four Archetype bridges and truck models are shown in Tables 8.3 to 8.6.

Step 2: Compute the annual consumed bridge fatigue life (N_C) *for each truck model*

The annual consumed bridge fatigue life for a particular truck model (axle configuration and weight) was determined using the expected truck traffic for this particular truck model in a year. The annual truck traffic (including all truck models) for a given bridge site can be estimated using the ADTT in NBI database (NBI 2012). The annual truck traffic for a given bridge site was then distributed to each truck model by the truck axle group distribution (Table 4.2) and the truck GVWs distribution (Table 4.6). For the sample calculation here, a 4000 ADTT value was used. Results for the sample calculation are shown in Table 9.1. Note that for specific bridges in South Carolina, the ADTT values from the NBI database were used (NBI 2012).

Table 9.1: Sample Calculation for Annual Consumed Bridge Fatigue Life.

Axle Group	Truck Type	ADTT	Percentage of Axle Group	Percentage of GVW1	Percentage of GVW2	Percentage of GVW3	Count for GVW1	Count for GVW2	Count for GVW3
2-Axle	A21	4000	8.84%	99.98%	0.01%	0.01%	128979	13	13
3-Axle	A31		5.70%	99.92%	0.06%	0.02%	N/A	N/A	17
	A32						83147	53	N/A
4-Axle	A41		4.60%	99.98%	0.01%	0.01%	22371	2	2
	A42						N/A	N/A	2
	A43						22371	2	N/A
	A44						N/A	N/A	2
	A45						22371	2	N/A
5-Axle	A51		78.49%	92.91%	4.66%	2.42%	N/A	N/A	27778
	A52						1064686	53439	N/A
6-Axle	A61		1.17%	95.54%	4.38%	0.08%	N/A	N/A	14
	A62						16265	746	N/A
7-Axle	A71		1.18%	94.25%	5.41%	0.34%	N/A	N/A	58
	A72						16209	931	N/A
8-Axle	A81		0.03%	32.98%	54.20%	12.82%	N/A	N/A	56
	A82						144	237	N/A

Step 3, Compute the annual bridge fatigue damage (D)

The annual bridge damage caused by a truck model is defined as the annual consumed fatigue life by this truck model (N_{Ci}) divided by the bridge fatigue life of this truck model (N_i). The total bridge fatigue damage (D) is the sum of fatigue damages from all truck models, as shown in Equation (9.1).

$$Fatigue\ Damage\ (D) = \frac{Consumed\ Fatigue\ Life}{Bridge\ Fatigue\ Life}$$

$$= \sum \left(\frac{N_{Ci,1}}{N_{i,1}} + \frac{N_{Ci,2}}{N_{i,2}} + \frac{N_{Ci,3}}{N_{i,3}} \right) \qquad (9.1)$$

where

$N_{Ci,1}$, $N_{Ci,2}$, $N_{Ci,3}$: number of loading cycles consumed for the *i-th* truck model with gross vehicle weight levels 1 to 3 (GVW1, GVW2, GVW3), respectively

$N_{i,1}$, $N_{i,2}$, $N_{i,3}$: allowable number of loading cycles for the *i-th* truck model with gross vehicle weight levels 1 to 3 (GVW1, GVW2, GVW3), respectively

i: Truck type

Note that the bridge fatigue damage (D) is a unitless quantity, where D equal to zero means no damage and D equal to unity means the particular bridge has used up its fatigue life (i.e. complete damage under repetitive fatigue loading). The results for all four Archetype bridges are listed in Tables 9.2 to 9.5.

Table 9.2: Sample Calculation for Annual Fatigue Damage of Archetype 1 Bridge.

Axle Group	Truck Type	Annual Fatigue Damage of GVW1	Annual Fatigue Damage of GVW2	Annual Fatigue Damage of GVW3	Annual Bridge Fatigue Damage
2-Axle	A21	1.15E-03	1.23E-07	1.26E-07	1.34%
3-Axle	A31	N/A	N/A	1.61E-07	
	A32	7.54E-04	4.89E-07	N/A	
4-Axle	A41	2.04E-04	2.19E-08	2.23E-08	
	A42	N/A	N/A	2.30E-08	
	A43	2.04E-04	2.21E-08	N/A	
	A44	N/A	N/A	2.21E-08	
	A45	2.01E-04	2.20E-08	N/A	
5-Axle	A51	N/A	N/A	2.88E-04	
	A52	9.75E-03	4.97E-04	N/A	
6-Axle	A61	N/A	N/A	1.36E-07	
	A62	1.50E-04	7.07E-06	N/A	
7-Axle	A71	N/A	N/A	6.24E-07	
	A72	1.48E-04	8.90E-06	N/A	
8-Axle	A81	N/A	N/A	5.91E-07	
	A82	1.35E-06	2.27E-06	N/A	

Table 9.3: Sample Calculation for Annual Fatigue Damage of Archetype 2 Bridge.

Axle Group	Truck Type	Annual Fatigue Damage of GVW1	Annual Fatigue Damage of GVW2	Annual Fatigue Damage of GVW3	Annual Bridge Fatigue Damage
2-Axle	A21	2.77E-04	1.03E-07	7.39E-07	2.62%
3-Axle	A31	N/A	N/A	1.48E-06	
	A32	9.65E-04	2.10E-06	N/A	
4-Axle	A41	2.44E-04	6.20E-08	2.94E-07	
	A42	N/A	N/A	5.79E-07	
	A43	2.12E-04	7.04E-08	N/A	
	A44	N/A	N/A	9.32E-07	
	A45	2.30E-04	6.26E-08	N/A	
5-Axle	A51	N/A	N/A	1.12E-02	
	A52	1.01E-02	1.81E-03	N/A	
6-Axle	A61	N/A	N/A	3.03E-06	
	A62	2.89E-04	7.51E-05	N/A	
7-Axle	A71	N/A	N/A	6.23E-05	
	A72	3.96E-04	2.23E-04	N/A	
8-Axle	A81	N/A	N/A	2.27E-05	
	A82	3.07E-06	4.83E-05	N/A	

Table 9.4: Sample Calculation for Annual Fatigue Damage of Archetype 3 Bridge.

Axle Group	Truck Type	Annual Fatigue Damage of GVW1	Annual Fatigue Damage of GVW2	Annual Fatigue Damage of GVW3	Annual Bridge Fatigue Damage
2-Axle	A21	1.89E-04	4.90E-08	1.05E-07	0.96%
3-Axle	A31	N/A	N/A	2.89E-07	
	A32	3.51E-04	6.67E-07	N/A	
4-Axle	A41	8.01E-05	2.36E-08	7.96E-08	
	A42	N/A	N/A	1.69E-07	
	A43	1.36E-04	4.07E-08	N/A	
	A44	N/A	N/A	8.59E-08	
	A45	6.74E-05	1.75E-08	N/A	
5-Axle	A51	N/A	N/A	1.87E-03	
	A52	5.95E-03	7.05E-04	N/A	
6-Axle	A61	N/A	N/A	1.41E-06	
	A62	7.76E-05	1.46E-05	N/A	
7-Axle	A71	N/A	N/A	3.35E-05	
	A72	1.05E-04	3.63E-05	N/A	
8-Axle	A81	N/A	N/A	6.89E-06	
	A82	7.89E-07	1.27E-05	N/A	

Table 9.5: Sample Calculation for Annual Fatigue Damage of Archetype 4 Bridge.

Axle Group	Truck Type	Annual Fatigue Damage of GVW1	Annual Fatigue Damage of GVW2	Annual Fatigue Damage of GVW3	Annual Bridge Fatigue Damage
2-Axle	A21	1.75E-04	6.85E-08	1.12E-07	1.15%
3-Axle	A31	N/A	N/A	3.77E-07	
	A32	3.70E-04	5.55E-07	N/A	
4-Axle	A41	7.82E-05	2.12E-08	6.83E-08	
	A42	N/A	N/A	1.68E-07	
	A43	1.23E-04	3.84E-08	N/A	
	A44	N/A	N/A	8.72E-08	
	A45	7.06E-05	2.32E-08	N/A	
5-Axle	A51	N/A	N/A	1.18E-03	
	A52	7.97E-03	1.05E-03	N/A	
6-Axle	A61	N/A	N/A	1.79E-06	
	A62	1.36E-04	2.89E-05	N/A	
7-Axle	A71	N/A	N/A	2.65E-05	
	A72	2.00E-04	5.60E-05	N/A	
8-Axle	A81	N/A	N/A	1.20E-05	
	A82	1.35E-06	1.31E-05	N/A	

Step 4, Determine the bridge replacement cost (C_R)

In this sample calculation, a replacement cost of $1 million dollars was assumed for all four Archetype bridges. The determination of the actual replacement cost for individual bridges in South Carolina is discussed in Chapter 7.

Step 5, Compute the annual bridge fatigue damage cost (C_D)

The annual bridge fatigue damage cost for a given bridge can be calculated by multiplying the annual bridge fatigue damage, *D* (computed in step 3) with the bridge replacement cost C_R (step 4).

$$\text{Damage Cost } (C_D) = C_R \times D \quad (9.2)$$

The results for this sample calculation, assuming a bridge replacement value of $1 million dollars, are shown in Table 9.6.

Table 9.6: Sample Calculation for Annual Bridge Fatigue Damage Cost.

Archetype Bridge	Bridge Replacement Cost (Dollar)	Annual Bridge Fatigue Damage	Annual Bridge Fatigue Damage Cost (Dollar)
A1	1,000,000	1.34%	13,374
A2	1,000,000	2.62%	26,185
A3	1,000,000	0.96%	9,639
A4	1,000,000	1.15%	11,492

<u>Annual Bridge Fatigue Damage Cost in South Carolina</u>

The results shown in Table 9.6 are for an assumed ADTT of 4000 and a bridge replacement cost of $1 million dollars. To compute the annual bridge fatigue damage cost for all bridges in South Carolina, the estimated average daily traffic data in the NBI database (NBI 2012) and the actual bridge replacement costs were used. This ADTT for each bridge was computed using the ADT (average daily traffic) multiplied with its truck percentage from the NBI database (NBI 2012). In the NBI database, the truck percentage for some bridges is listed as zero. For those bridges with a zero truck percentage, a nominal ADTT equal to 1% of the ADT was assumed. It should be noted that the ADT entries in the NBI database were not all recorded for the same year. A 2% annual increase in ADT was used to adjust and normalize the ADT of all bridges to year 2011.

Table 9.7 shows the total bridge replacement costs and the associated damage costs for the four Archetype bridges. The total replacement cost for those bridges that were not represented by the four Archetype bridges, shown as "Others" in Table 9.7, was determined by subtracting the sum of the replacement costs of the four Archetype bridges from the total bridge replacement cost in South Carolina determined in Chapter 7 (i.e. \$9.332 billion 2011 US dollars). Also shown in Table 9.7 are the annual damage cost ratios, expressed as fraction of the total replacement cost for each Archetype bridge group. The damage cost ratio for each Archetype was computed as the annual bridge fatigue damage cost divided by the total replacement cost of the Archetype bridge group. For "others" bridges their total annual bridge fatigue damage cost was estimated using the average damage cost ratio of the four Archetype bridges multiplied with their bridge replacement cost (\$5.727 billion dollars). As shown in Table 9.7, the total annual bridge fatigue damage cost in South Carolina was found to be approximately \$29.35 million dollars (2011 US Dollar).

Table 9.7: Annual Bridge Fatigue Damage Cost in South Carolina.

Archetype Bridge	Bridge Replacement Cost (Dollar)	Annual Bridge Fatigue Damage Cost (Dollar)	Annual Damage Cost Ratio
A1	1,619,338,243	3,365,836	0.0021
A2	1,203,787,124	5,554,071	0.0046
A3	584,554,296	1,640,698	0.0028
A4	197,142,364	627,899	0.0032
Others	5,727,329,116	18,161,514	0.0032
All	9,332,151,143	29,350,017	

Annual Bridge Maintenance Cost in South Carolina

As stated previously, the total bridge cost included both fatigue damage cost and maintenance cost. The annual bridge maintenance cost was obtained from the SCDOT maintenance cost schedule for the period of July 2010 to June 2011 (SCDOT 2012c). The total annual cost for activities related to the bridge maintenance (i.e. exclude bridge replacement) was found to be approximately equal to \$6.445 million dollars (Equation 9.3). The complete maintenance schedule and cost breakdowns can be found in Appendix D.

$$\text{Annual Bridge Maintenance Cost } (C_M) = 6.445 \; million \; dollars \tag{9.3}$$

Annual Bridge Cost in South Carolina

The annual bridge cost in South Carolina was computed by adding up the annual bridge fatigue damage cost and the annual bridge maintenance cost (Equation 9.4).

$$Annual \; Bridge \; Cost \; (C) = C_D + C_M \tag{9.4}$$

where

C_D: is the annual bridge fatigue damage cost in South Carolina

C_M: is the annual bridge maintenance cost in South Carolina

The results are given in Table 9.8. It was found that the total annual bridge cost in South Carolina is approximately \$35.795 million dollars (2011 US Dollar). Recall that the study from Ohio Department of Transportation found a much larger annual bridge cost, which was approximately \$308 million dollars, as discussed in Chapter 2 (ODOT 2009). The is because the ODOT study calculated annual bridge cost of all bridges in

Ohio by assuming 1/75 of their total replacement cost were consumed each year (i.e. based on the target bridge design life of 75 years specified in AASHTO) while in this research annual bridge cost for a given bridge was calculated by multiplying its annual bridge fatigue damage with its bridge replacement cost.

Table 9.8: Annual Bridge Cost in South Carolina.

Annual Fatigue Damage Cost (Dollar)	Annual Maintenance Cost (Dollar)	Total Annual Cost (Dollar)
29,350,017	6,445,420	35,795,437

CHAPTER TEN

OVERWEIGHT TRUCK BRIDGE COST

Introduction

In order to identify the impact of overweight trucks on the bridge network, the annual bridge cost was allocated to overweight trucks in South Carolina based on the damage contribution of overweight trucks and the percentage of overweight trucks in the overall truck population. For the purpose of setting fee structure for operating overweight trucks, the unit costs (cost per mile) of overweight trucks of different axle configurations and gross weights are also computed using the vehicle miles traveled (VMT) of individual truck models.

Annual Bridge Cost Allocated to Overweight Trucks

Similar to the total annual bridge cost calculation, the annual bridge cost allocated to overweight trucks included two types of costs, namely the bridge fatigue and maintenance costs. Allocation of bridge coast to overweight trucks was based on the damage contribution of the overweight trucks. The truck models with either gross vehicle weight levels 2 and 3 (GVW2 and GVW3) are considered to be overweight trucks.

The allocation of bridge damage cost was carried out using the damage contribution of the overweight trucks:

$$C_{D,O} = \frac{D_{GVW2} + D_{GVW3}}{D} \times C_D \tag{10.1}$$

where

$C_{D,O}$: is the annual bridge damage cost allocated to all overweight trucks

D_{GVW2}: is the annual bridge fatigue damage caused by all GVW2 trucks

D_{GVW3}: is the annual bridge fatigue damage caused by all GVW3 trucks

D: is the total annual bridge fatigue damage

C_D: is the annual bridge fatigue damage cost.

In the sample fatigue damage calculation for Archetype 1 to 4 bridges shown in Table 9.2 to Table 9.5, the overweight trucks are the GVW2 and GVW3 trucks and the normal or non-overweight weight trucks are the GVW1 truck. Table 10.1 to Table 10.4 present the breakdowns of the damage contributions of normal and overweight trucks for Archetypes 1 to 4 bridges, respectively. The annual fatigue damages caused by the normal weight trucks (Table 10.1 to Table 10.4) were the same as the annual fatigue damages of the GVW1 trucks in Table 9.2 to Table 9.5. The annual bridge fatigue damage by overweight trucks was obtained by summing up the annual fatigue damage caused by the GVW2 and GVW3 trucks in Table 9.2 to Table 9.5. The percent contribution of overweight trucks to the total annual fatigue damage was computed by dividing the damage caused by overweight trucks (GVW2 and GVW3) by the total annual bridge fatigue damage. As can be seen from Table 10.1 to Table 10.4, overweight trucks are much more detrimental to prestressed concrete girder bridges than to reinforced concrete slab bridges. The overweight trucks make up of 5.71% of the overall truck population. However, they contribute to approximately 6% of the damages of the reinforced concrete slab bridges and 20% to 50% of the damages of prestressed concrete girder bridges (see Table 10.1 to Table 10.4).

Table 10.1: Percentage of Damage by Overweight Trucks for Archetype 1 Bridge.

Axle Group	Annual Fatigue Damage by Normal Weight Trucks	Annual Fatigue Damage by Overweight Trucks	Total Annual Bridge Fatigue Damage	Percentage Damage by Overweight Trucks
2-Axle	1.15E-03	2.49E-07		
3-Axle	7.54E-04	6.49E-07		
4-Axle	6.09E-04	1.33E-07		
5-Axle	9.75E-03	7.85E-04	1.34%	6.02%
6-Axle	1.50E-04	7.20E-06		
7-Axle	1.48E-04	9.53E-06		
8-Axle	1.35E-06	2.86E-06		

Table 10.2: Percentage of Damage by Overweight Trucks for Archetype 2 Bridge.

Axle Group	Annual Fatigue Damage by Normal Weight Trucks	Annual Fatigue Damage by Overweight Trucks	Total Annual Bridge Fatigue Damage	Percentage Damage by Overweight Trucks
2-Axle	2.77E-04	8.41E-07		
3-Axle	9.65E-04	3.59E-06		
4-Axle	6.86E-04	2.00E-06		
5-Axle	1.01E-02	1.30E-02	2.62%	51.42%
6-Axle	2.89E-04	7.81E-05		
7-Axle	3.96E-04	2.85E-04		
8-Axle	3.07E-06	7.10E-05		

Table 10.3: Percentage of Damage by Overweight Trucks for Archetype 3 Bridge.

Axle Group	Annual Fatigue Damage by Normal Weight Trucks	Annual Fatigue Damage by Overweight Trucks	Total Annual Bridge Fatigue Damage	Percentage Damage by Overweight Trucks
2-Axle	1.89E-04	1.54E-07	0.96%	27.83%
3-Axle	3.51E-04	9.56E-07		
4-Axle	2.83E-04	4.16E-07		
5-Axle	5.95E-03	2.58E-03		
6-Axle	7.76E-05	1.60E-05		
7-Axle	1.05E-04	6.99E-05		
8-Axle	7.89E-07	1.96E-05		

Table 10.4: Percentage of Damage by Overweight Trucks for Archetype 4 Bridge.

Axle Group	Annual Fatigue Damage by Normal Weight Trucks	Annual Fatigue Damage by Overweight Trucks	Total Annual Bridge Fatigue Damage	Percentage Damage by Overweight Trucks
2-Axle	1.75E-04	1.81E-07	1.15%	20.57%
3-Axle	3.70E-04	9.32E-07		
4-Axle	2.72E-04	4.06E-07		
5-Axle	7.97E-03	2.22E-03		
6-Axle	1.36E-04	3.07E-05		
7-Axle	2.00E-04	8.25E-05		
8-Axle	1.35E-06	2.51E-05		

The annual bridge fatigue damage costs allocated to overweight trucks are summarized in Table 10.5. It was found that the total annual fatigue damage cost due to overweight trucks is approximately \$8.449 million dollars which is 28.8% of the estimated total annual bridge fatigue damage cost (\$29.35 million dollars, 2011 US Dollar) in South Carolina. While overweight trucks consist of approximately 5.7% of the truck population, they are responsible for 28.8% of the bridge damage cost.

Table 10.5: Annual Bridge Fatigue Damage Cost Allocated to Overweight Trucks.

Archetype Bridge	Annual Bridge Fatigue Damage Cost (Dollar)	Percentage of Damage by Overweight Trucks	Annual Bridge Fatigue Damage Cost Allocated to Overweight Trucks (Dollar)
A1	3,365,836	6.02%	202,685
A2	5,554,071	51.42%	2,855,694
A3	1,640,698	27.83%	456,543
A4	627,899	20.57%	129,149
Others	18,161,514	26.46%	4,805,202
Total	29,350,017		8,449,273

The allocation of the maintenance cost to the overweight trucks was carried out by percentage of the overweight truck in the total truck population (Equation 10.2).

$$C_{M,O} = \frac{N_{GVW2} + N_{GVW3}}{N_{GVW1} + N_{GVW2} + N_{GVW3}} \times C_M \tag{10.2}$$

where:

$C_{M,O}$: is the annual bridge maintenance cost allocated to the overweight trucks

$N_{GVW1}, N_{GVW2}, N_{GVW3}$: are the number of trucks for gross vehicle weight levels GVW1, GVW2 and GVW3, respectively

C_M: is the total annual bridge maintenance cost

According to the NBI database, the total ADT for all bridges in South Carolina was 45,706,454 and the total ADTT for all bridges was 4,316,773 (i.e. 9.44% of traffic was truck). Using the overweight trucks distribution data shown in Table 4.2 and Table 4.6, it was found that around 246,491 of the total ADTT were from the overweight trucks (GVW2 truck and GVW3 truck). Therefore, using Equation (10.3), the annual bridge maintenance cost allocated to the overweight trucks was determined to be (Table 10.6).

$$C_{M,O} = \frac{246{,}491}{45{,}706{,}454} \times 6{,}445{,}420 = 34{,}760\ Dollars \qquad (10.3)$$

Table 10.6: Annual Bridge Maintenance Cost Allocated to Overweight Trucks.

Annual Bridge Maintenance Cost (Dollar)	Annual Bridge Maintenance Cost by Overweight Trucks (Dollar)
6,445,420	34,760

The total annual bridge cost allocated to the overweight trucks was calculated in Equation (10.4) and the results are summarized in Table 10.7. The annual bridge cost caused by the overweight trucks is approximately $8.484 million dollars (2011 US Dollar).

$$C_O = C_{D,O} + C_{M,O} \qquad (10.4)$$

where

C_O: is the total annual bridge cost allocated to overweight trucks

$C_{D,O}$: is the annual bridge damage cost allocated to overweight trucks

$C_{M,O}$: is the annual bridge maintenance cost allocated to overweight trucks

Table 10.7: Annual Bridge Cost Allocated to Overweight Trucks.

Annual Bridge Fatigue Damage Cost Allocated to Overweight Trucks (Dollar)	Annual Bridge Maintenance Cost Allocated to Overweight Trucks (Dollar)	Annual Bridge Cost Allocated to Overweight Trucks(Dollar)
8,449,273	34,760	8,484,033

Overweight Trucks Bridge Cost per Mile

There are multiple ways to set the fee structure for overweight permits. A rational method would be to base it on the overweight trucks' unit cost (cost per mile) and then use the mileages travelled of overweight trucks to determine their overweight fee. Because the mileages travelled by overweight trucks include not only bridges but also other infrastructures such as pavement, the overweight trucks' unit cost was calculated as per mile of road travelled, instead of per bridge length travelled. Since trucks with different weights and axle configurations cause different levels of damages, the overweight trucks bridge costs per mile in this research were computed by axle group.

The overweight trucks bridge cost per mile for each axle group was computed as follow:

$$C_{Pj,O} = \frac{C_{Oj}}{VMT_{j,O}} \tag{10.5}$$

where

C_{Oj}: Daily bridge cost allocated to overweight trucks in each axle group

$VMT_{j,O}$: Daily VMT (vehicle miles travelled) by overweight trucks in the axle group being considered.

j: Axle group

The daily bridge cost allocated to overweight trucks in each axle group consisted of two parts: the daily fatigue damage cost and the daily maintenance cost. The allocation of daily fatigue damage cost to each axle group was carried out using the fatigue damage of overweight trucks in each axle group divided by the total fatigue damage of overweight trucks.

Firstly, the daily bridge fatigue damage cost allocated to overweight trucks was calculated by dividing the annual fatigue costs of overweight trucks (Table 10.5) by 365 days. The daily bridge fatigue damage costs caused by overweight trucks are grouped by bridge Archetype and are summarized in Table 10.8.

Table 10.8: Daily Bridge Fatigue Damage Cost Allocated to Overweight Trucks.

Archetype Bridge	Annual Bridge Fatigue Damage Cost Allocated to Overweight Trucks (Dollar)	Daily Bridge Fatigue Damage Cost Allocated to Overweight Trucks (Dollar)
A1	202,685	555
A2	2,855,694	7,824
A3	456,543	1,251
A4	129,149	354
Others	4,805,202	13,165
Total	8,449,273	23,149

Secondly, the above daily costs were then distributed to each axle group based on the percentage of overweight trucks fatigue damage of each axle group in the total overweight trucks fatigue damage as shown in Table 10.9 to Table 10.12. As seen in these tables, because the 5-axle trucks are the most common trucks, the collective fatigue damages caused by the 5-axle overweight trucks are the highest for all four Archetype bridges.

Table 10.9: Daily Bridge Fatigue Damage Cost Allocated to Overweight Trucks in Each Axle Group for Archetype 1 Bridge.

Axle Group	Annual Fatigue Damage by Overweight Trucks	Total Annual Fatigue Damage by Overweight Trucks	Overweight Damage Distribution	Overweight Damage Cost (Dollar)
2-Axle	2.49E-07	0.08%	0.03%	0.17
3-Axle	6.49E-07		0.08%	0.45
4-Axle	1.33E-07		0.02%	0.09
5-Axle	7.85E-04		97.44%	541.08
6-Axle	7.20E-06		0.89%	4.97
7-Axle	9.53E-06		1.18%	6.57
8-Axle	2.86E-06		0.36%	1.97

Table 10.10: Daily Bridge Fatigue Damage Cost Allocated to Overweight Trucks in Each Axle Group for Archetype 2 Bridge.

Axle Group	Annual Fatigue Damage by Overweight Trucks	Total Annual Fatigue Damage by Overweight Trucks	Overweight Damage Distribution	Overweight Damage Cost (Dollar)
2-Axle	8.41E-07	1.35%	0.01%	0.49
3-Axle	3.59E-06		0.03%	2.08
4-Axle	2.00E-06		0.01%	1.16
5-Axle	1.30E-02		96.73%	7567.67
6-Axle	7.81E-05		0.58%	45.38
7-Axle	2.85E-04		2.12%	165.78
8-Axle	7.10E-05		0.53%	41.25

Table 10.11: Daily Bridge Fatigue Damage Cost Allocated to Overweight Trucks in Each Axle Group for Archetype 3 Bridge.

Axle Group	Annual Fatigue Damage by Overweight Trucks	Total Annual Fatigue Damage by Overweight Trucks	Overweight Damage Distribution	Overweight Damage Cost (Dollar)
2-Axle	1.54E-07	0.27%	0.01%	0.07
3-Axle	9.56E-07		0.04%	0.45
4-Axle	4.16E-07		0.02%	0.19
5-Axle	2.58E-03		96.01%	1200.90
6-Axle	1.60E-05		0.60%	7.47
7-Axle	6.99E-05		2.61%	32.59
8-Axle	1.96E-05		0.73%	9.12

Table 10.12: Daily Bridge Fatigue Damage Cost Allocated to Overweight Trucks in Each Axle Group for Archetype 4 Bridge.

Axle Group	Annual Fatigue Damage by Overweight Trucks	Total Annual Fatigue Damage by Overweight Trucks	Overweight Damage Distribution	Overweight Damage Cost (Dollar)
2-Axle	1.81E-07	0.24%	0.01%	0.03
3-Axle	9.32E-07		0.04%	0.14
4-Axle	4.06E-07		0.02%	0.06
5-Axle	2.22E-03		94.08%	332.89
6-Axle	3.07E-05		1.30%	4.60
7-Axle	8.25E-05		3.49%	12.35
8-Axle	2.51E-05		1.06%	3.76

In the above tables, the total annual fatigue damages by overweight trucks were computed using the results shown in Table 10.1 to Table 10.4 for the four Archetype bridges. For example, the annual fatigue damage to Archetype 1 bridges by all truck traffic was estimated to be 1.34% and overweight trucks responsible for 6.02% of the 1.34% damage (Table 10.1). Hence, the annual fatigue damage to Archetype 1 bridges by only the overweight trucks was 0.08% (1.34% x 6.02%) (see Table 10.9). The overweight damage distribution for each axle group was computed by dividing the overweight damage of respective axle group by the total overweight damage. Using the overweight damage distribution of axle groups, the daily bridge fatigue damage cost allocated to overweight trucks in each axle group was then computed (Table 10.13). For the other bridges (i.e. other than Archetypes 1 to 4), an average ratio from the four Archetype bridges for each axle group was used to compute the daily damage cost contribution of each axle group. Table 10.13 summarizes the total daily overweight damage cost for all bridges by axle group.

Table 10.13: Daily Bridge Fatigue Damage Cost Allocated to Overweight Trucks in Each Axle Group.

Axle Group	A1 Overweight Damage Cost (Dollar)	A2 Overweight Damage Cost (Dollar)	A3 Overweight Damage Cost (Dollar)	A4 Overweight Damage Cost (Dollar)	Other Overweight Damage Cost (Dollar)	Total Overweight Damage Cost (Dollar)
2-Axle	0.17	0.49	0.07	0.03	1.66	2.42
3-Axle	0.45	2.08	0.45	0.14	6.00	9.12
4-Axle	0.09	1.16	0.19	0.06	2.11	3.62
5-Axle	541.08	7567.67	1200.90	332.89	12646.83	22289.38
6-Axle	4.97	45.38	7.47	4.60	110.97	173.39
7-Axle	6.57	165.78	32.59	12.35	309.30	526.60
8-Axle	1.97	41.25	9.12	3.76	88.06	144.17

Recall that the *annual* bridge maintenance cost allocated to overweight trucks was estimated to be 34,760 dollars (Table 10.6), so, the daily bridge maintenance cost allocated to overweight trucks was 95 dollars (34,758/365). This daily maintenance cost was then allocated to each axle group based on the overweight truck proportion of each axle group. In Table 10.14, the axle group percentages were determined from weigh-in-motion data (see Table 4.2) and the percentages of GVW2+GVW3 (i.e. overweight trucks) were calculated from Table 4.6. The percentage of overweight trucks for each axle group was calculated as the axle group percentage multiplied by the percentage of GVW2+GVW3. The relative distribution of overweight trucks for each axle group was obtained using the percentage of overweight trucks for each axle group (column 4 in Table 10.14) divided by the total percentage of overweight trucks (5.71%). The daily bridge maintenance costs of overweight trucks by axle group are presented in Table 10.15. Because the 5-axle trucks are the most recorded trucks in weigh-in-motion data (Table 4.2 and Table 4.6), the daily bridge maintenance costs of 5-axle overweight trucks is the highest.

Table 10.14: Overweight Trucks Relative Distribution.

Axle Group	Axle Group Percentage	Percentage of GVW2+GVW3	Percentage of Overweight Trucks	Total Percentage for Overweight Trucks	Overweight Trucks Relative Distribution
2	8.84%	0.02%	0.002%	5.71%	0.03%
3	5.70%	0.08%	0.005%		0.08%
4	4.60%	0.02%	0.001%		0.02%
5	78.49%	7.09%	5.563%		97.42%
6	1.17%	4.46%	0.052%		0.91%
7	1.18%	5.75%	0.068%		1.19%
8	0.03%	67.02%	0.020%		0.35%

Table 10.15: Daily Bridge Maintenance Cost Allocated to Overweight Trucks in Each Axle Group.

Axle Group	Daily Bridge Maintenance Cost Allocated to Overweight Trucks (Dollar)	Overweight Trucks Relative Distribution	Daily Bridge Maintenance Cost Allocated to Overweight Trucks in Each Axle Group (Dollar)
2-Axle	95	0.03%	0.03
3-Axle		0.08%	0.08
4-Axle		0.02%	0.02
5-Axle		97.42%	92.77
6-Axle		0.91%	0.87
7-Axle		1.19%	1.13
8-Axle		0.35%	0.34

Table 10.16 shows the daily bridge cost allocated to overweight trucks in each axle group (C_{Oj}) which is calculated by adding up the daily bridge fatigue damage cost and the daily bridge maintenance cost allocated to overweight trucks in each axle group.

Table 10.16: Daily Bridge Cost Allocated to Overweight trucks in Each Axle Group.

Axle Group	Daily Bridge Cost Allocated to Overweight Trucks in Each Axle Group (Dollar)
2-Axle	2
3-Axle	9
4-Axle	4
5-Axle	22382
6-Axle	174
7-Axle	528
8-Axle	145

Table 10.17 shows the daily vehicle miles travelled (VMT) of overweight trucks in South Carolina categorized by axle group ($VMT_{j,O}$). The VMT for each road was calculated using the ADTT (average daily truck traffic) of the respective road multiplied by the road length. Then the total VMT was computed by adding up the VMT of the road network. The total VMT was further divided into the VMT of overweight truck by axle group (Table 10.17).

Table 10.17: Overweight VMT Distribution in Each Axle Group.

Axle Group	Daily Overweight VMT
2-Axle	495
3-Axle	1341
4-Axle	267
5-Axle	1558294
6-Axle	14569
7-Axle	18187
8-Axle	4853

Finally, the overweight truck bridge cost per mile by each axle group was calculated using Equation (10.5), by dividing the daily cost (Table 10.16) by the daily VMT (Table 10.17). The overweight truck bridge costs per mile by axle group are shown in Table 10.18 It can be seen that the overweight trucks bridge cost per mile increases as the number of axles increases. This is because trucks with more axles are generally heavier than trucks with fewer axles. Unlike pavement where the damage is mainly governed by the load of single axle, for bridges, gross vehicle weight has a more significant impact on the bridge damage than the axle load alone. An example calculation for damage cost per trip is also provided in Table 10.18. Assuming a trip length of 100 miles, the corresponding cost for each axle group can easily be determined by multiplying the trip length by the cost per mile (see Table 10.18). The results shown in Table 10.18 can be used for further analysis for establishing an overweight permit fee structure based on vehicle mile travelled.

Table 10.18: Overweight Trucks Bridge Cost per Mile in Each Axle Group.

Axle Group	Overweight Trucks Bridge Cost per Mile (Dollar)	Overweight Trucks Bridge Cost per Trip (100 miles)
2-Axle	0.005	0.496
3-Axle	0.007	0.686
4-Axle	0.014	1.363
5-Axle	0.014	1.436
6-Axle	0.012	1.196
7-Axle	0.029	2.902
8-Axle	0.030	2.977

CHAPTER ELEVEN

SUPER-LOAD TRUCK BRIDGE COST

Introduction

It has been observed that the relationship between damage and truck weight is highly nonlinear. The damages to bridges caused by trucks with extreme high loadings, referred herein as *super-load*, can be significantly higher than that of the trucks with their weights between the legal weight limit and the maximum weight limit. In this study, super-load means the truck gross vehicle weight is more than the maximum weight limit allowed by the South Carolina Department of Transportation (SCDOT 2012a). This chapter presents the development of functional relationships between bridge cost per mile and gross vehicle weight for estimating the damage cost of super-load trucks.

Super-load Trucks Bridge Cost per Mile

The first step in developing the functional relationship between bridge cost per mile and gross vehicle weight was to compute bridge costs per mile for each axle group for the three distant weigh levels, namely GVW1, GVW2, and GVW3. The methodology used to compute the super-load trucks bridge cost per mile for each gross vehicle weight level and axle group was the same as the one used to determine the overweight trucks bridge cost per mile in Chapter 10. The estimated costs per mile by weight and axle group are shown in Table 11.1 and detailed calculations can be found in Appendix E. Figure 11.1 to figure 11.7 show super-load trucks bridge cost per mile as functions of gross vehicle weight (GVW) and axle groups. A nonlinear exponential trend line was fitted to the three data points of each axle group, which corresponded to the three GVW levels (i.e. GVW1, GVW2 and GVW3):

$$C = c_1 e^{(c_2 \times GVW)} \tag{11.1}$$

where, C is the bridge cost per mile (in 2011 USD) and GVW is gross vehicle weight of the truck in kips. c_1 and c_2 are coefficients determined through least-square regression. The fitted coefficients are shown in Figures 11.1 to 11.7.

Table 11.1: GVW1, GVW2 and GVW3 Trucks Bridge Cost per Mile in Each Axle Group.

Axle Group	GVW1 Trucks Bridge Cost per Mile (Dollar)	GVW2 Trucks Bridge Cost per Mile (Dollar)	GVW3 Trucks Bridge Cost per Mile (Dollar)
2-Axle	0.0012	0.0020	0.0053
3-Axle	0.0021	0.0048	0.0088
4-Axle	0.0020	0.0041	0.0234
5-Axle	0.0023	0.0052	0.0320
6-Axle	0.0028	0.0107	0.0308
7-Axle	0.0036	0.0223	0.1650
8-Axle	0.0036	0.0238	0.0559

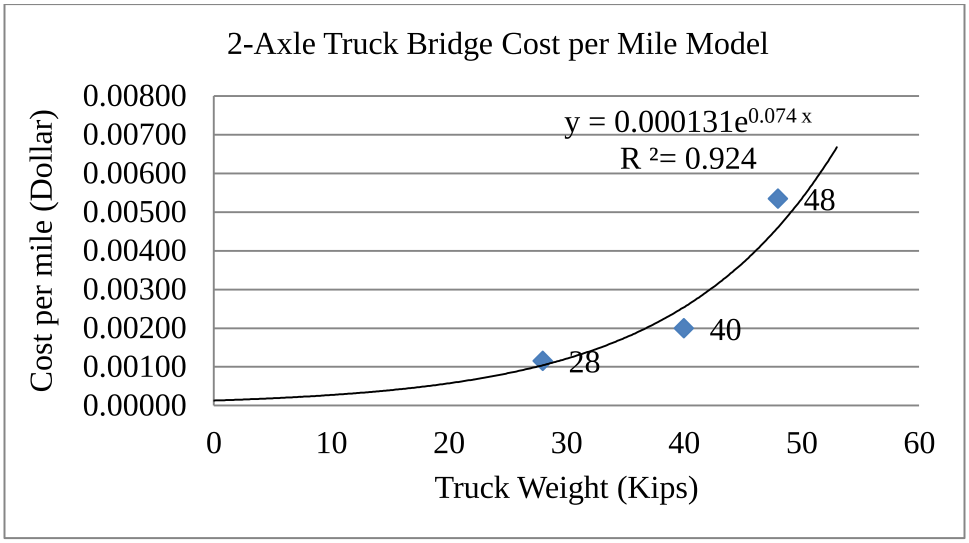


Figure 11.1: 2-Axle Truck Bridge Cost per Mile Model.

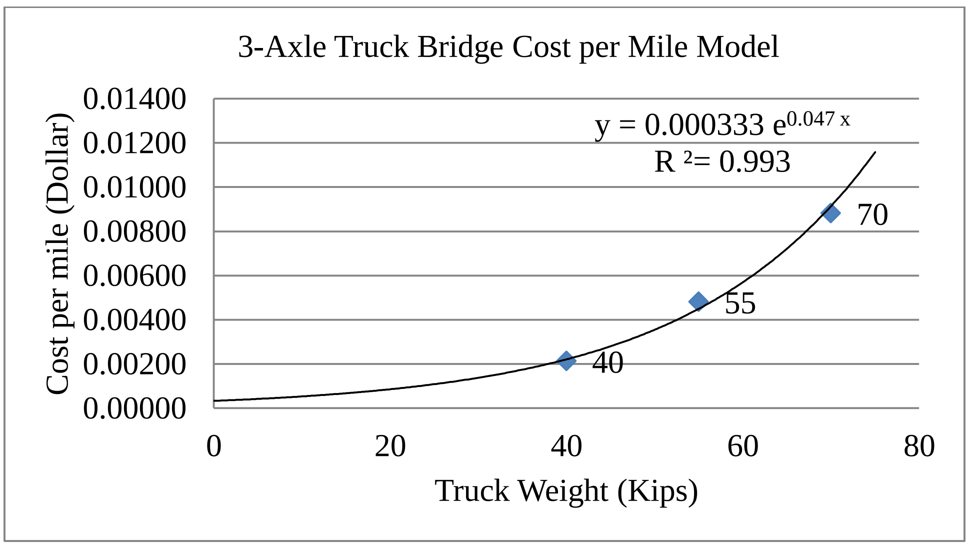


Figure 11.2: 3-Axle Truck Bridge Cost per Mile Model.

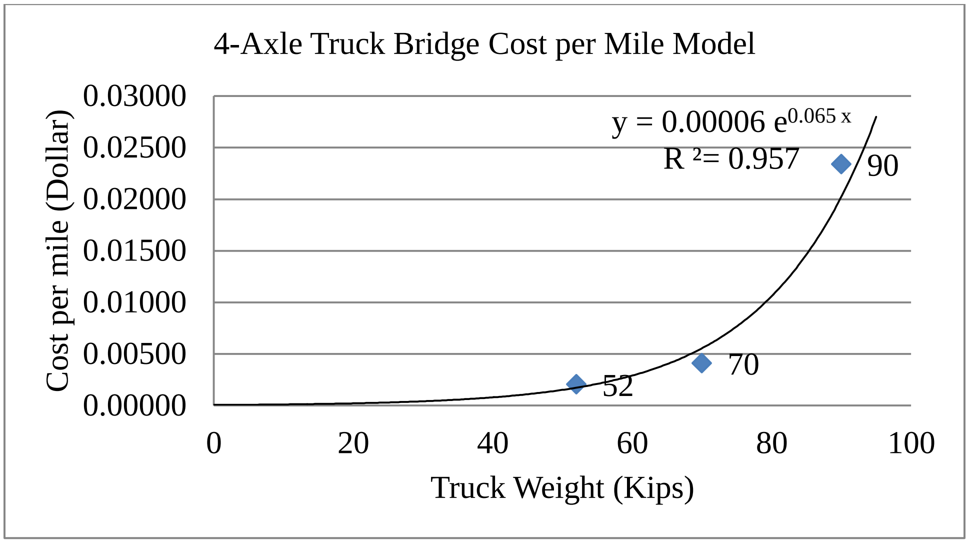


Figure 11.3: 4-Axle Truck Bridge Cost per Mile Model.

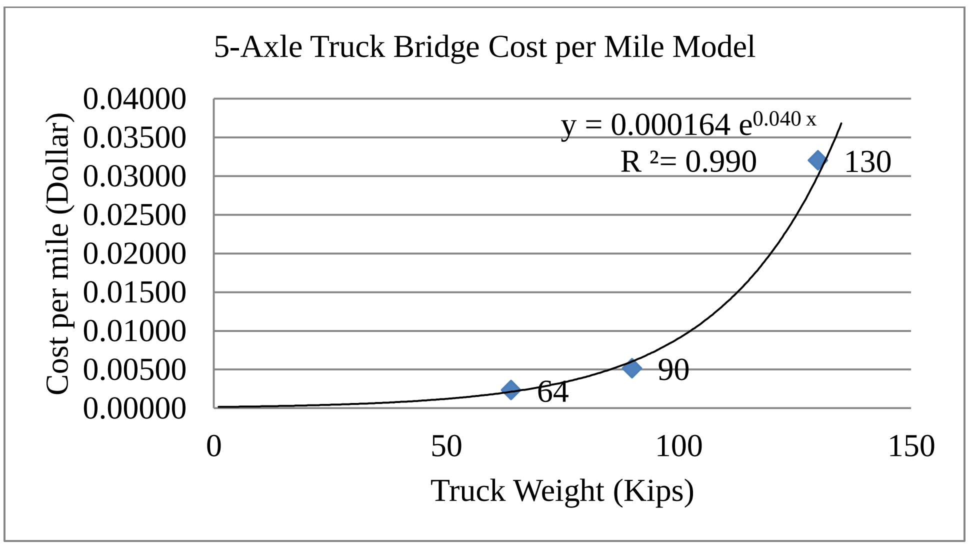


Figure 11.4: 5-Axle Truck Bridge Cost per Mile Model.

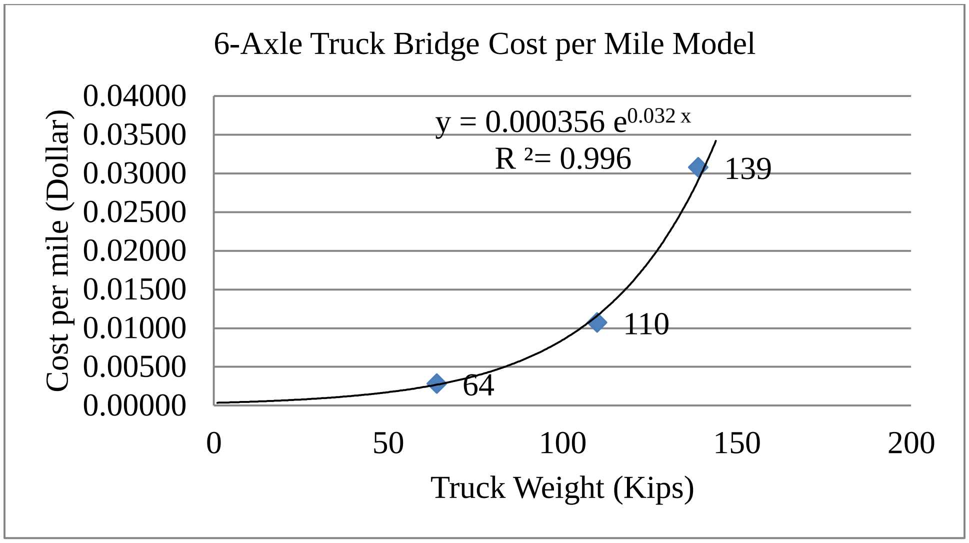


Figure 11.5: 6-Axle Truck Bridge Cost per Mile Model.

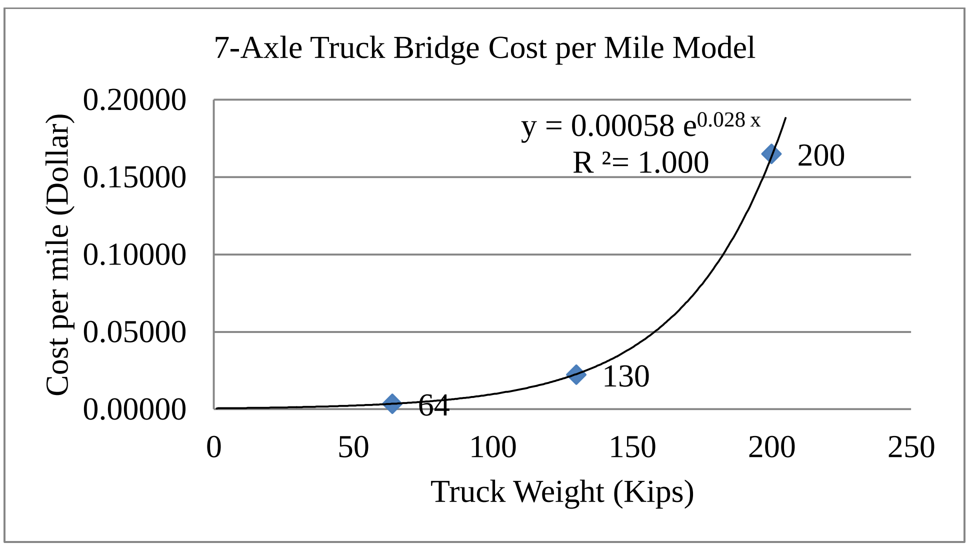


Figure 11.6: 7-Axle Truck Bridge Cost per Mile Model.

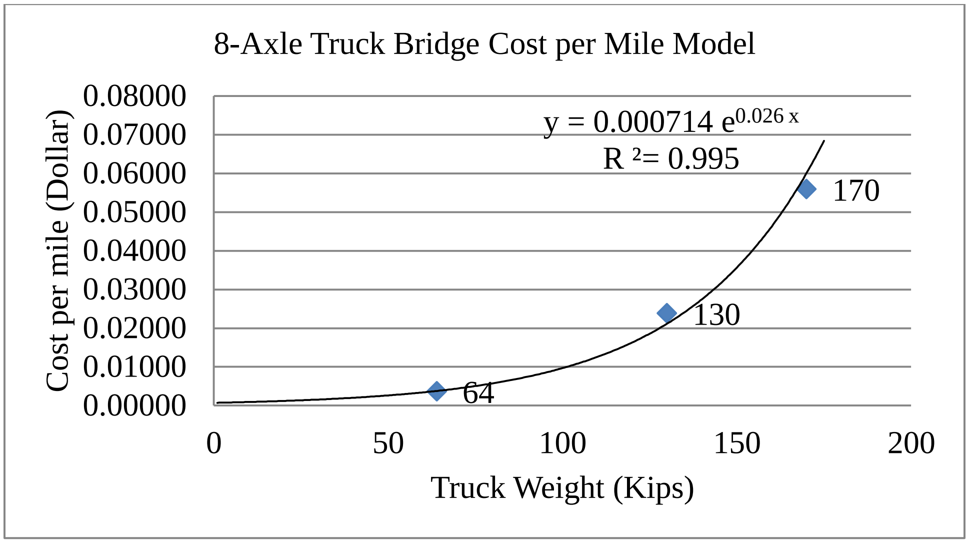


Figure 11.7: 8-Axle Truck Bridge Cost per Mile Model.

As can be seen, the relationship between cost per mile and truck weight is highly nonlinear. For example, for the most common 5-axle trucks (Figure 11.4), a 44% increase in gross vehicle weight from 90 kips to 130 kips results in 515% increase in damage cost per mile (from 0.0052 dollar to 0.032 dollar per mile).

Using the cost per mile models developed for different axle groups, the cost of any arbitrary gross vehicle weight including that of the super-load trucks can be estimated. Given the limited data used to derive the cost per mile and truck weigh relationships, these models must be used with caution.

While the R^2 values of the fitted curves are very close to 1 which mean the goodness-of-fit of the trend lines are extremely well, this is because three data points were used to estimate two coefficients, which typically will yield good agreement. More accurate curves might be obtained if more data points are utilized.

Since GVW1 and GVW3 are the lower and upper limits for each curve, respectively, application of the cost models for truck weights within these limits is considered to be accurate. However, great care in application of these models is necessary if the truck gross weight is outside of the limits.

The costs per mile of three levels of super-loading were computed in this study. These three super-loads are defined as follows:

Super load 1: GVW2 + 25% x (GVW3-GVW2)

Super load 2: GVW2 + 50% x (GVW3-GVW2)

Super load 3: GVW2 + 75% x (GVW3-GVW2)

Note that GVW2 and GVW3 correspond to maximum weight limits and maximum considered weight for each axle group trucks, respectively. Using the bridge cost per miles models, the costs of these three super-loads for all axle groups were calculated (Table 11.2). The results presented here may be used by the South Carolina Department of Transportation to establish or adjust the fee structure for operating super-load trucks.

Table 11.2: Super-Load Trucks Bridge Cost per Mile in Each Axle Group.

Axle Group	Super-Load	Vehicle Gross Weight (Kips)	Bridge Cost per Mile (Dollar)
2-Axle	1	42	0.00295
	2	44	0.00342
	3	46	0.00397
3-Axle	1	59	0.00537
	2	63	0.00641
	3	66	0.00765
4-Axle	1	75	0.00768
	2	80	0.01061
	3	85	0.01466
5-Axle	1	100	0.00904
	2	110	0.01350
	3	120	0.02017
6-Axle	1	117	0.01464
	2	125	0.01843
	3	132	0.02320
7-Axle	1	148	0.03720
	2	165	0.06093
	3	183	0.09979
8-Axle	1	140	0.02748
	2	150	0.03567
	3	160	0.04630

CHAPTER TWELVE

SUMMARY AND CONCLUSIONS

The impact of overweight trucks on existing bridges has been an urgent concern for many states including South Carolina. This research quantified the annual bridge cost in South Carolina caused by trucks and overweight trucks. The annual bridge cost quantified in this study included two parts: damage cost and maintenance cost. Truck models with varying gross vehicle weight and axle configuration which are representative of trucks found on the South Carolina highway routes were developed using weigh-in-motion data, size and weight inspection violations data and SCDOT overweight truck permit data. Trucks with 5 axles were found to be the most common overweight trucks and very few 2-axle, 3-axle and 4-axle overweight trucks were recorded.

Since reinforced concrete and prestressed concrete bridge are the predominant bridge types in South Carolina, four Archetype bridges were developed to represent reinforced concrete slab and prestressed concrete girder bridges in South Carolina. Finite element (FE) models for the four Archetype bridges were created using the LS-DYNA program.

Bridge cost models for predicting the replacement costs of bridges in South Carolina were developed. The bridge cost models were developed as a function of either total structural length or total structural area. Using the bridge replacement cost models, the total replacement cost for all bridges in South Carolina was estimated to be $9.332 billion dollars (2011 US Dollar).

Annual bridge fatigue damage was estimated using the stress ranges calculated from the FE analyses. The total annual bridge damage cost in South Carolina was estimated to be $29.35 million dollars (2011 US Dollar) which is approximately 0.315% of the total bridge replacement cost. The annual bridge maintenance cost ($6.445 million dollar) was added to the annual bridge damage cost to obtain the total annual bridge cost in South Carolina, which was estimated to be approximately $35.795 million dollars.

Based on the damage contribution and percentage of overweight trucks in the overall truck population, the annual bridge cost allocated to overweight trucks (including bridge damage costs and bridge maintenance cost) was found to be $8.484 million dollars. It should be noted that the overweight trucks made up of 5.71% of the overall truck population but they contributed to approximately 23.7% of the total bridge cost. Compared to reinforced concrete bridges in which 6% of the bridge damage cost was attributed to overweight trucks, the impact of overweight trucks on prestressed concrete bridges was much more significant in which 20% to 50% of the bridge damages were due to overweight trucks.

In addition, unit costs (cost per mile) of overweight trucks and super-load trucks of different axle configurations and gross weights were also computed. It was found that the overweight trucks bridge cost per mile increases as the number of axles increases. This is because trucks with more axles are generally heavier than trucks with fewer axles. It was observed that the relationship between bridge damage cost per mile and truck weight was highly nonlinear. A set of nonlinear exponential bridge damage cost per mile models, expressed in terms of the gross vehicle weight and axle numbers, were developed for estimating the damage cost of truck of any arbitrary weight, in particular, for the super-load trucks. These damage cost models may be used by the state department of transportation to set the overweight permit fee structure.

Contribution and Suggestion of Use

This study developed a comprehensive methodology to estimate not only bridge damage but also the cost associated with bridge damage, including total annual bridge cost and unit costs (cost per mile) for trucks of different weights and axle configurations in South Carolina. This methodology starts with building representative truck models and archetype bridges in South Carolina. While the results presented in this study are applicable to only South Carolina, the methodology can easily be applied to study the impact of overweight trucks on bridge networks for other states or regions.

As part of this study, a methodology was developed for creating surrogate truck models to represent the truck population in South Carolina. These surrogate truck models were created to characterize the variability in axle configurations and truck weight distributions. While the truck models were specifically developed for South Carolina truck population, one can apply the procedure presented in this study to develop sets of suitable truck models to represent the truck population in other states.

Bridge cost models for bridges in South Carolina were developed. The bridge cost models were developed as a function of either the total structural length or total structural area; hence, they can be easily applied to estimate the replacement costs of bridges in South Carolina. Due to the proximity of surrounding states and similarities in bridge construction of the Southeast region, the replacement cost models may be applicable to other surrounding states such as Georgia and North Carolina,.

Suggestion for Further Study

In this study, the weigh-in-motion data used to calculate the bridge fatigue was from one location only. The damage cost contribution of overweight trucks estimated in this research hinges heavily on the accuracy of the observed percent overweight trucks via the weigh-in-motion data. In order to obtain a more accurate estimate of the bridge damage due to overweight truck, the weigh-in-motion data for more locations should be used.

In this study, four Archetype bridges representative of reinforce concrete slab and prestressed concrete bridges were used to estimate the damage cost of all bridges in South Carolina. The damages of steel bridges were inferred using the damage results of these concrete bridges. To obtain a more accurate estimate of the actual damages of steel bridges, it is recommended that future study with detailed models for steel bridges be conducted.

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APPENDICES

Appendix A

<u>Weigh-In-Motion Data</u>

Weigh-in-motion data was recorded from Nov 25, 2011 to May 25, 2012 at StGeorge 1 site.

Table A.1: Weign-In-Motion Data.

GVW (tonnes)			Number of Trucks for Each Vehicle Class											Total
			Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13	Class 14	Class 15	
0	-	10	49,413	4,195	1	27,060	792	5	1,065	0	0	0	7	82,538
10	-	20	1,854	2,061	40	22,880	120,144	3,366	3,223	18	19	0	0	153,605
20	-	30	1	410	206	2,848	116,651	5,704	8,398	16	58	0	16	134,308
30	-	40	0	2	21	19	202,984	3,727	1,928	49	57	0	2	208,789
40	-	50	0	0	5	0	209	404	0	16	62	0	1	697
50	-	60	0	0	0	0	1	109	0	6	59	0	1	176
60	-	70	0	0	0	0	0	6	0	0	36	0	0	42
70	-	75	0	0	0	0	0	0	0	0	5	0	5	10
75	-	80	0	0	0	0	0	0	0	0	2	0	2	4
80	-	85	0	0	0	0	0	0	0	0	2	0	4	6
85	-	90	0	0	0	0	0	0	0	0	0	0	15	15
90	-	100	0	0	0	0	0	0	0	0	0	0	29	29

100	-	110	0	0	0	0	0	0	0	0	0	0	2	2
110	-	120	0	0	0	0	0	0	0	0	0	0	0	0
120	-	130	0	0	0	0	0	0	0	0	0	0	0	0
130	+		0	0	0	0	0	0	0	0	0	0	0	0
Total			51,268	6,668	273	52,807	440,781	13,321	14,614	105	300	0	84	580,221
Percentage for vehicle class			8.84 %	1.15 %	0.05 %	9.10 %	75.97 %	2.30 %	2.52 %	0.02 %	0.05 %	0.00 %	0.01 %	

Distribution Parameters and Figures

Table A.2: Distribution Parameters.

Vehicle Class	Axle Group	w	u	k
5	2-Axle	10	18.606	2.749
6	3-Axle	10	19.971	1.470
7	4-Axle	10	42.134	3.837
8	3-Axle	10	19.496	1.435
	4-Axle	10	19.496	1.435
9	5-Axle	10	50.020	2.130
10	6-Axle	10	46.758	2.100
	7-Axle	10	46.758	2.100
11	5-Axle	10	40.304	2.547
12	6-Axle	10	63.124	2.530
13	7-Axle	10	85.344	2.145
	8-Axle	10	85.344	2.145

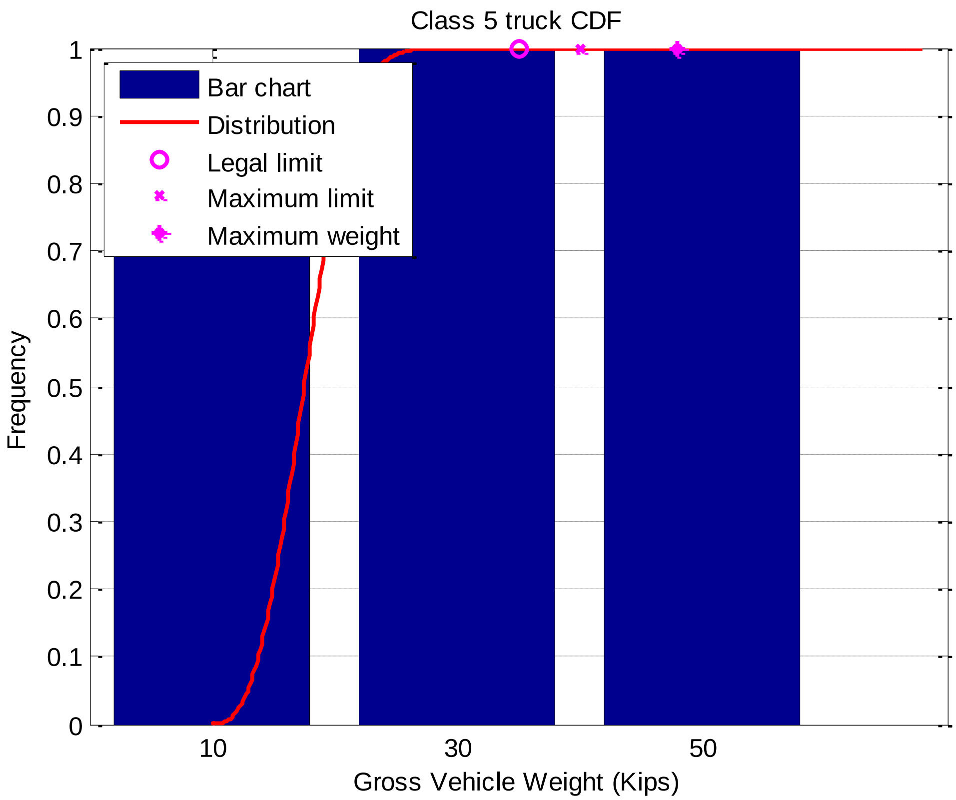


Figure A.1: Class 5 Truck CDF.

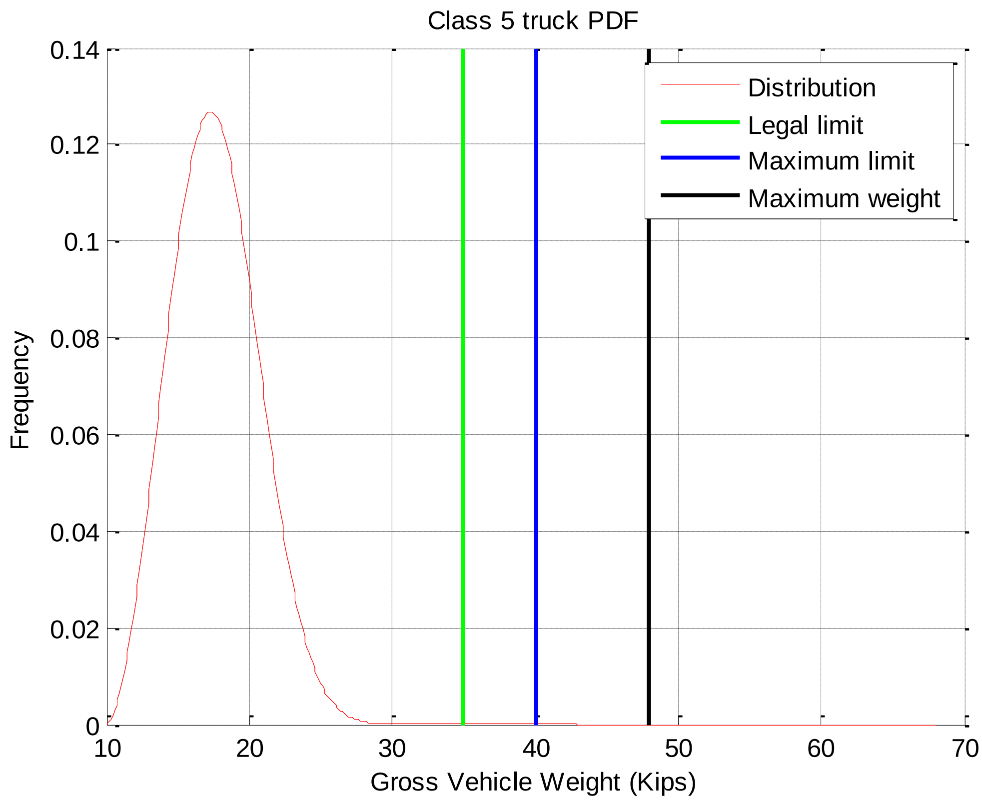


Figure A.2: Class 5 Truck PDF.

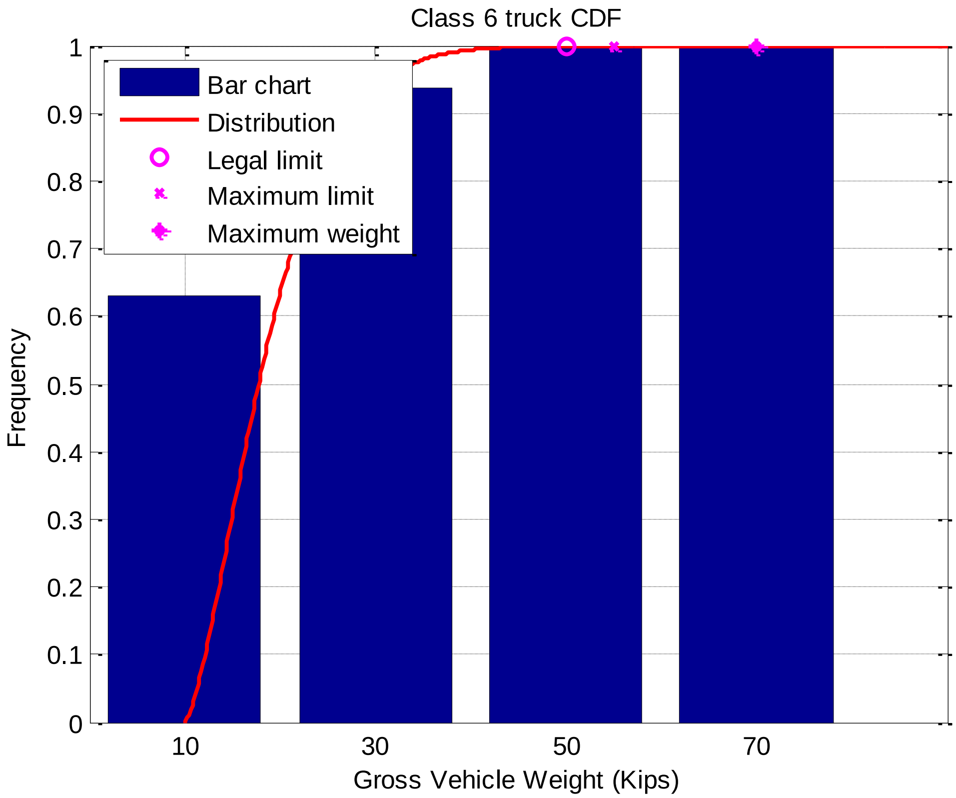


Figure A.3: Class 6 Truck CDF.

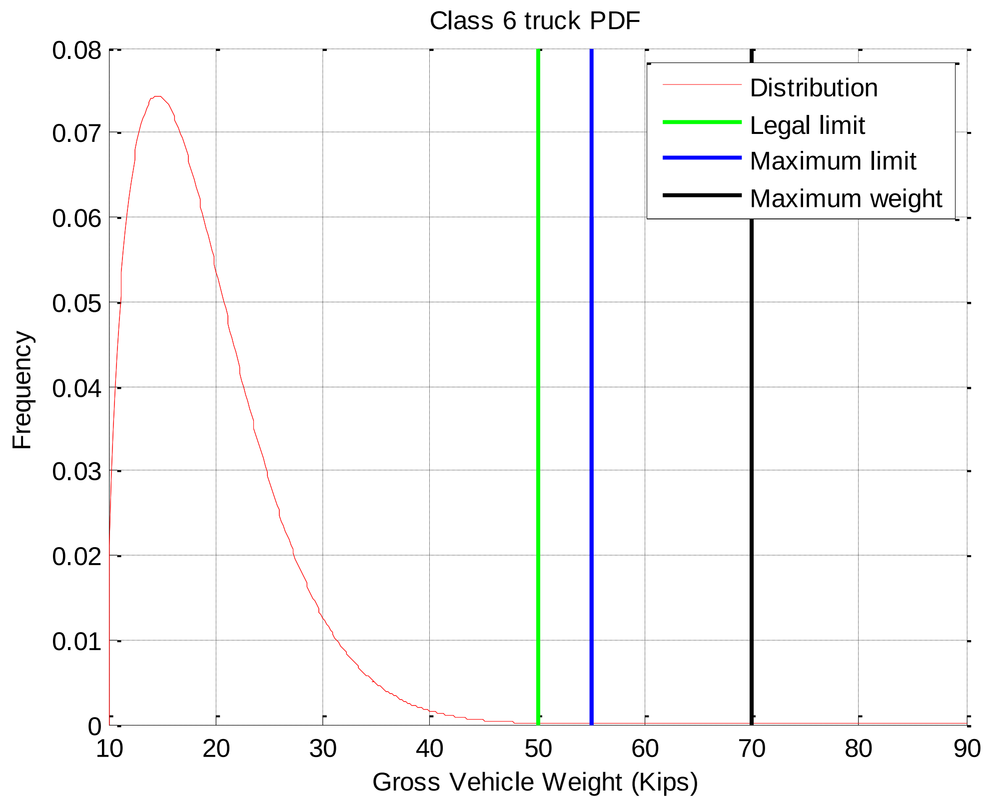


Figure A.4: Class 6 Truck PDF.

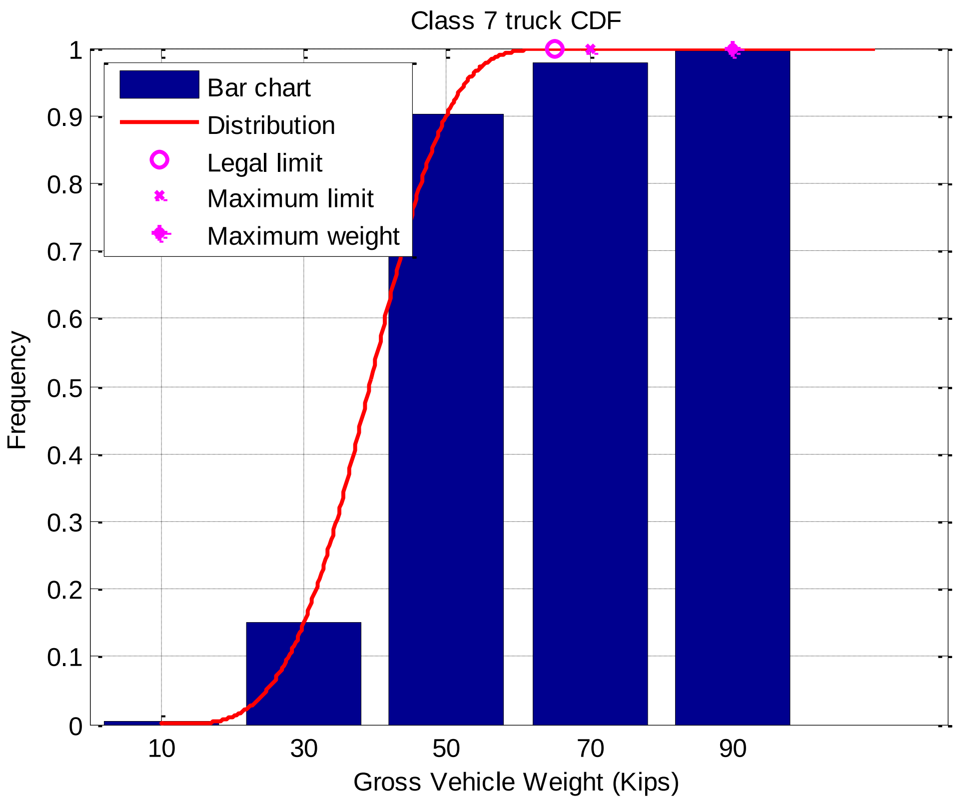


Figure A.5: Class 7 Truck CDF.

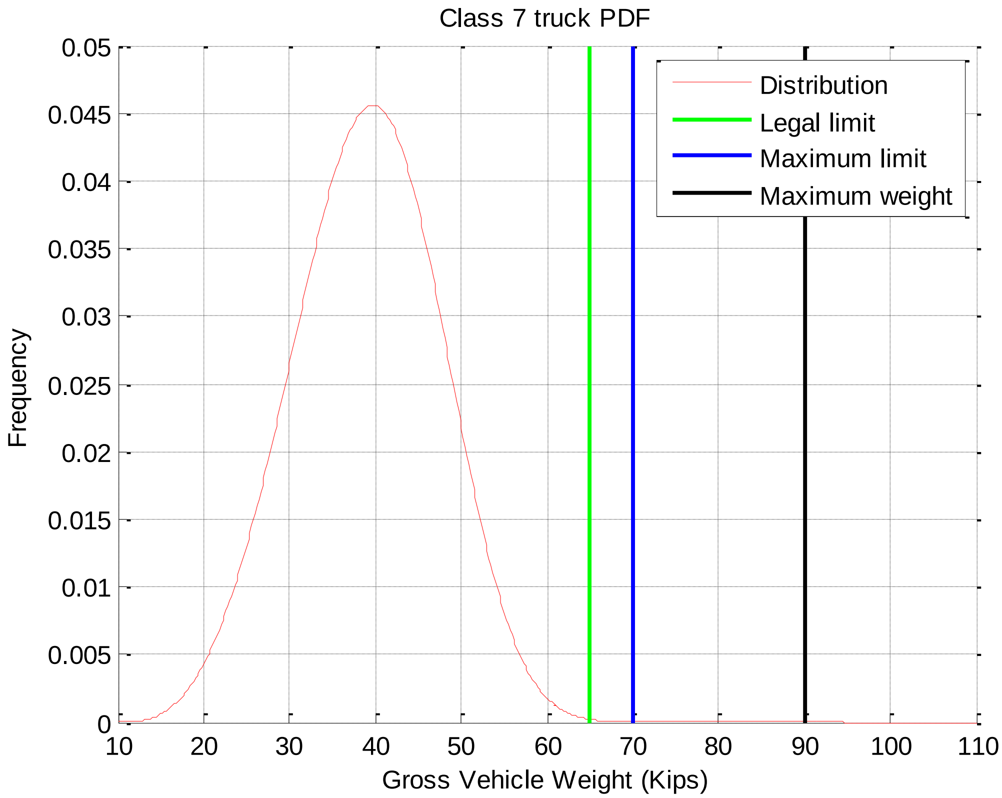


Figure A.6: Class 7 Truck PDF.

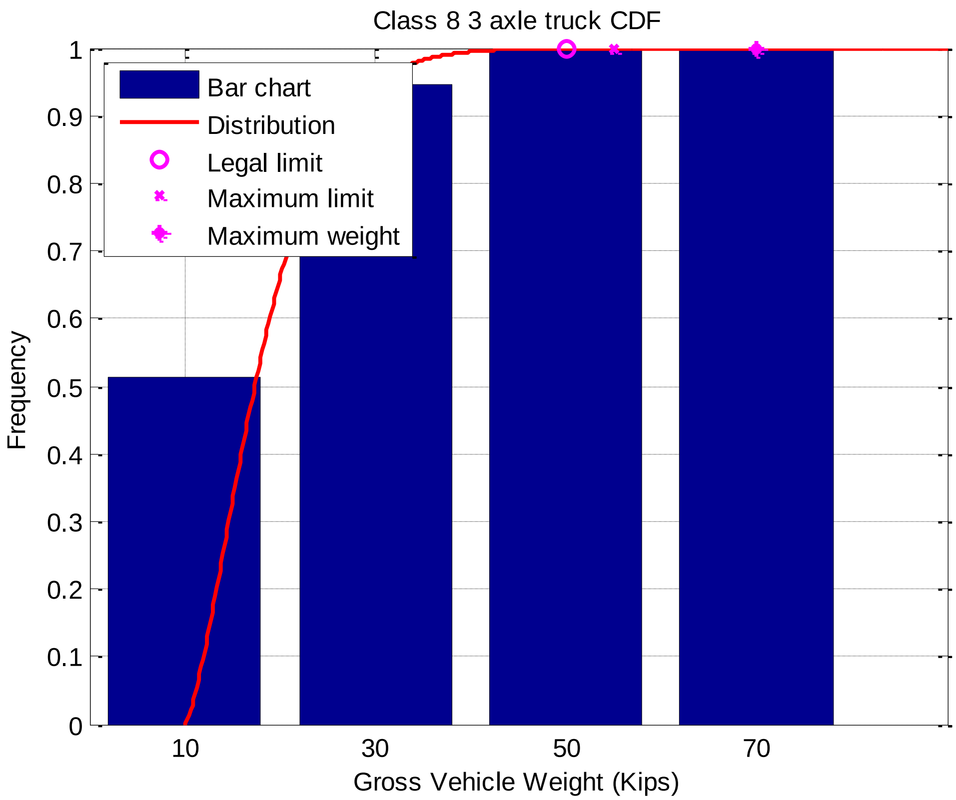


Figure A.7: Class 8 3 Axle Truck CDF.

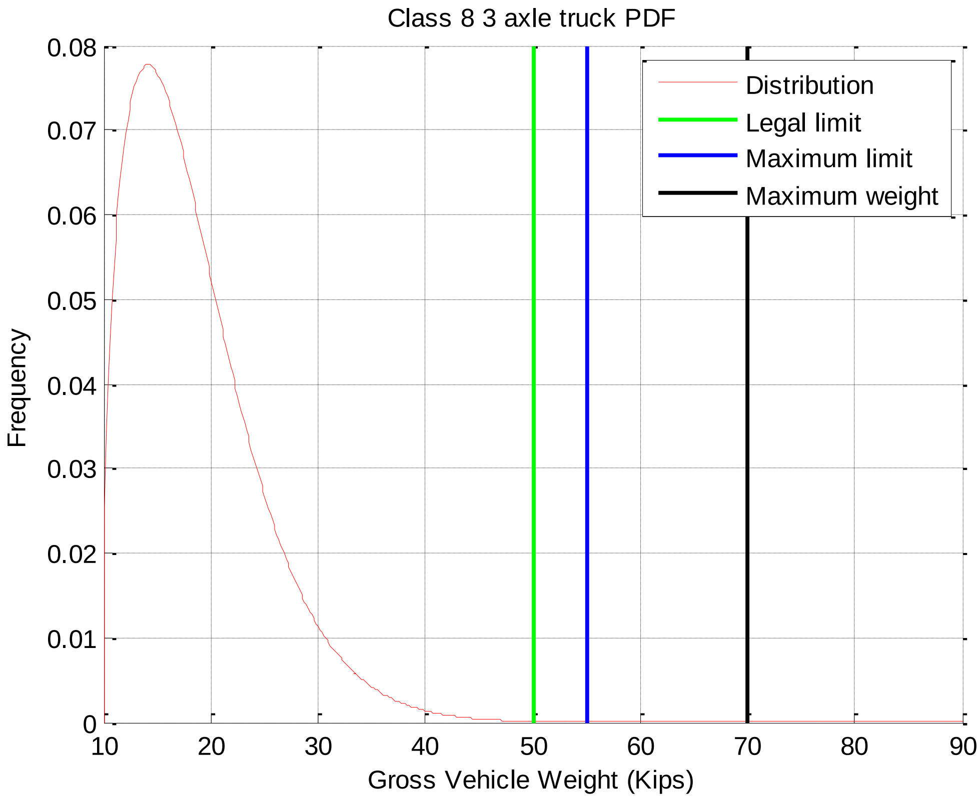


Figure A.8: Class 8 3 Axle Truck PDF.

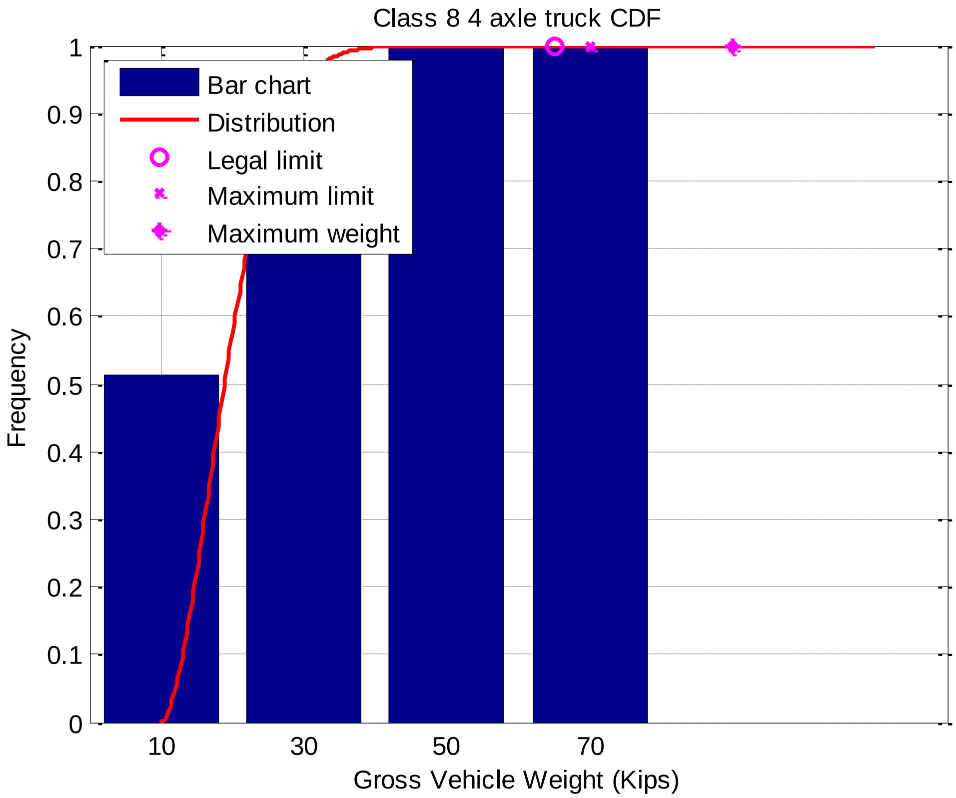


Figure A.9: Class 8 4 Axle Truck CDF.

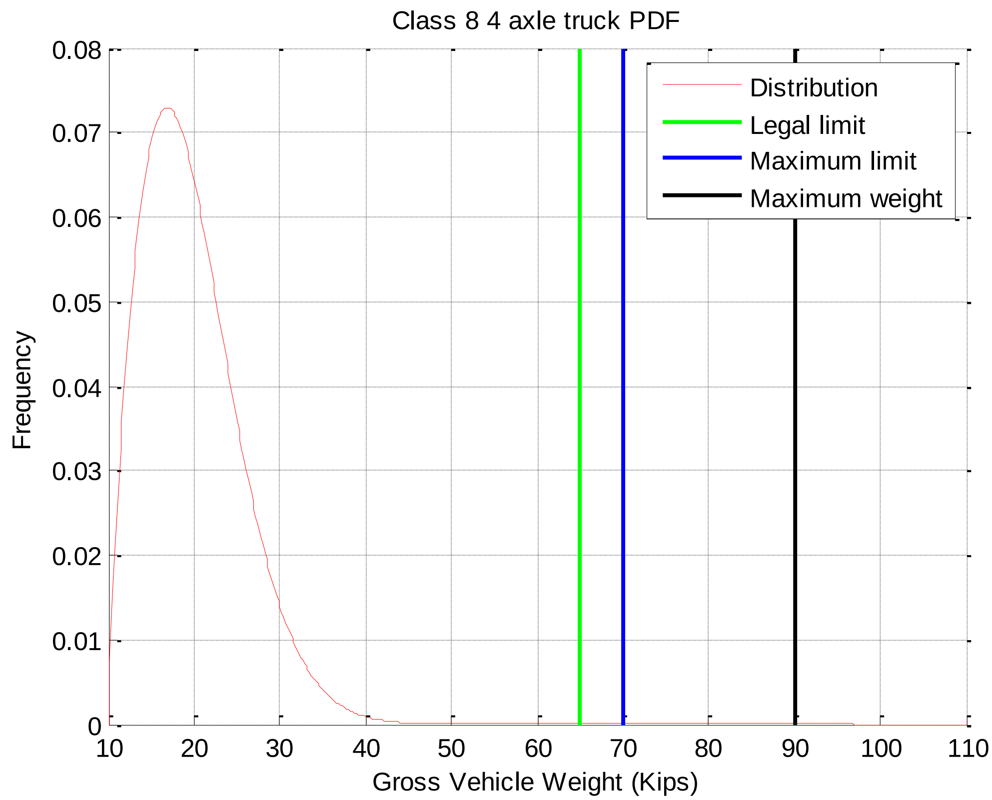


Figure A.10: Class 8 4 Axle Truck PDF.

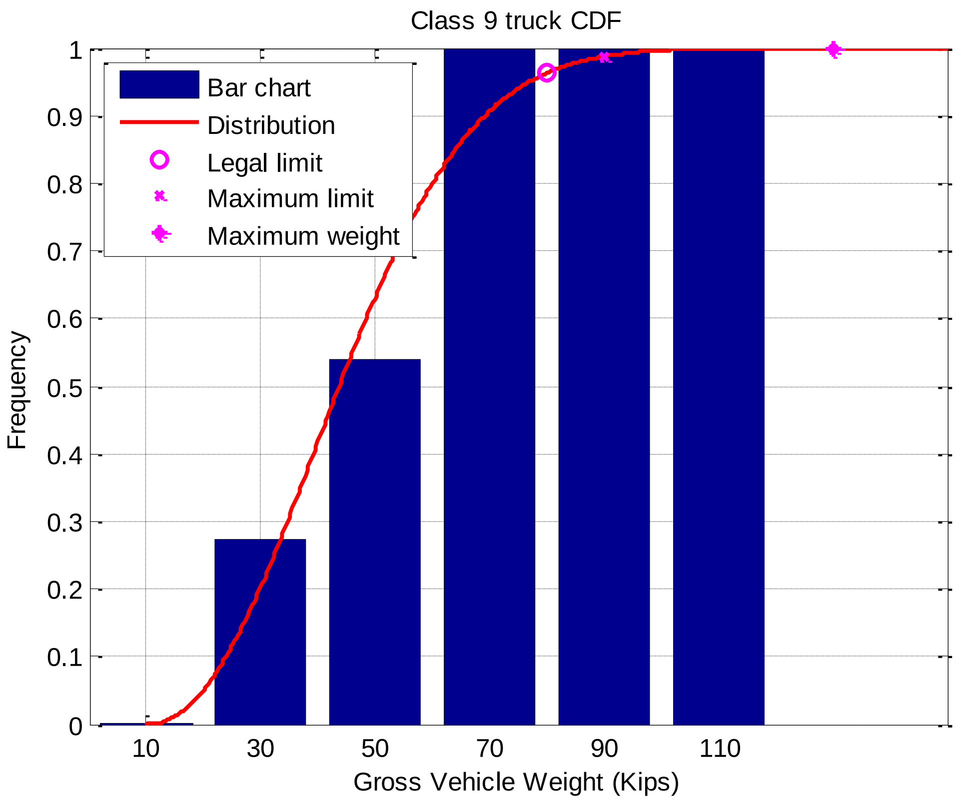


Figure A.11: Class 9 Truck CDF.

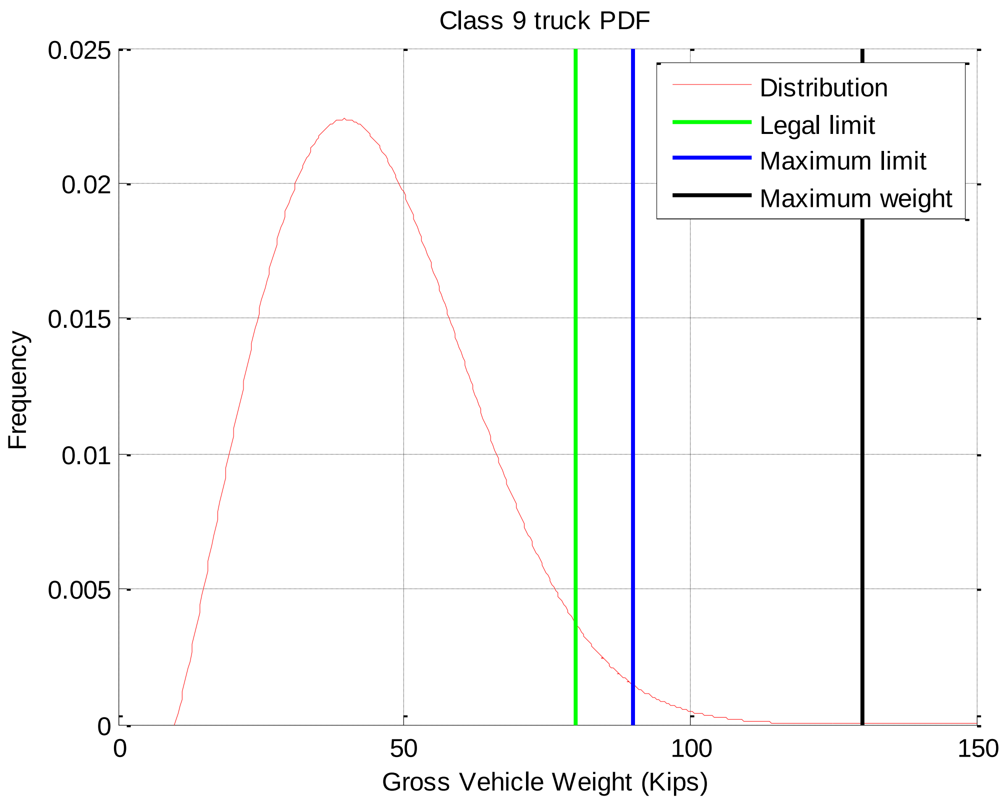


Figure A.12: Class 9 Truck PDF.

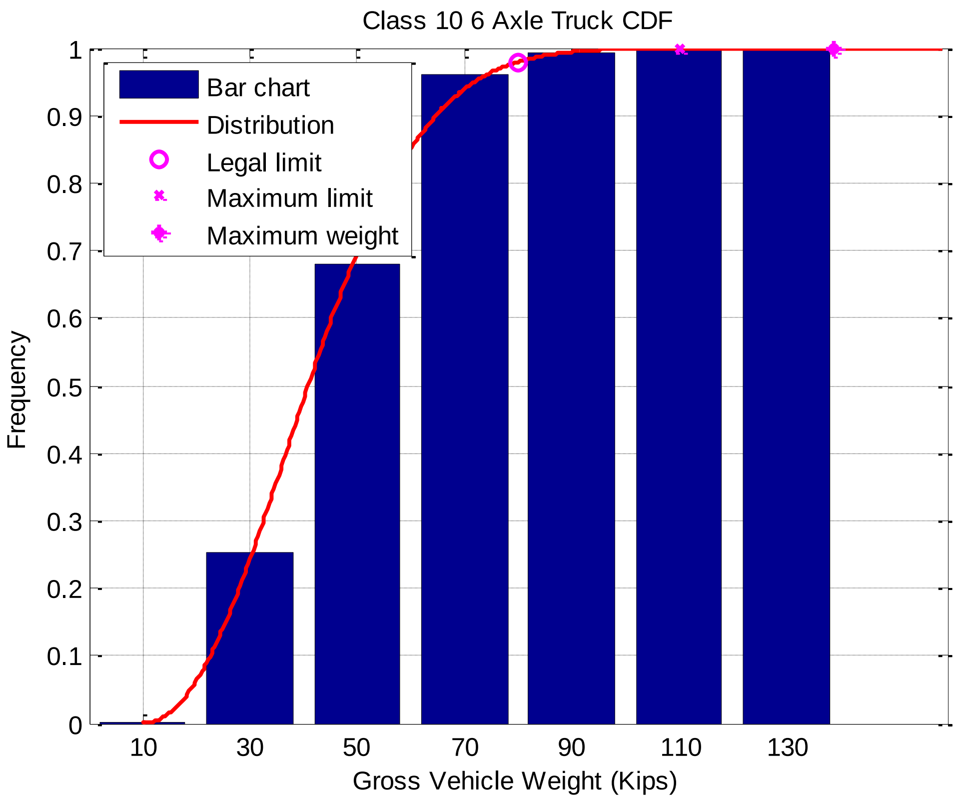


Figure A.13: Class 10 6 Axle Truck CDF.

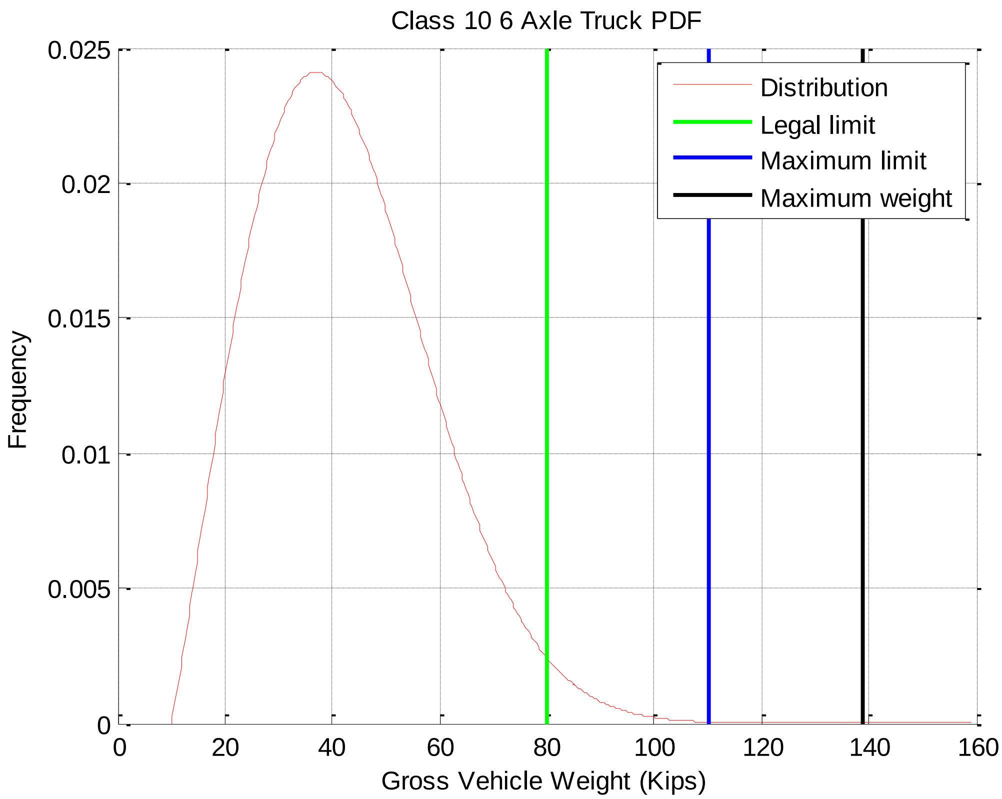


Figure A.14: Class 10 6 Axle Truck PDF.

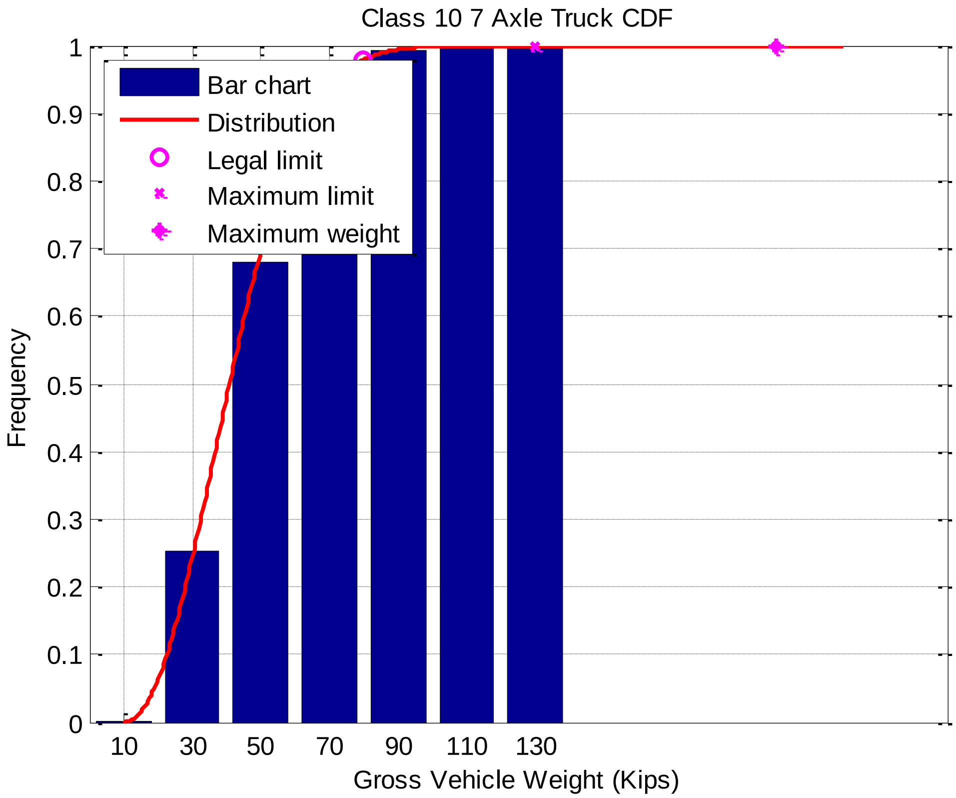


Figure A.15: Class 10 7 Axle Truck CDF.

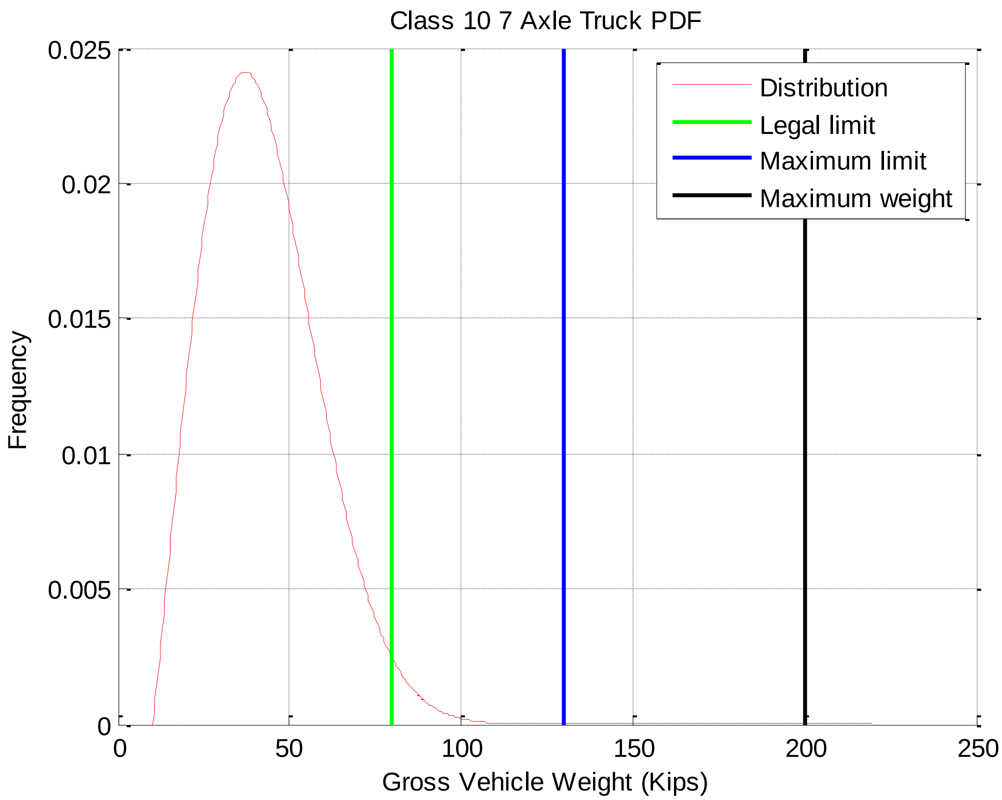


Figure A.16: Class 10 7 Axle Truck PDF.

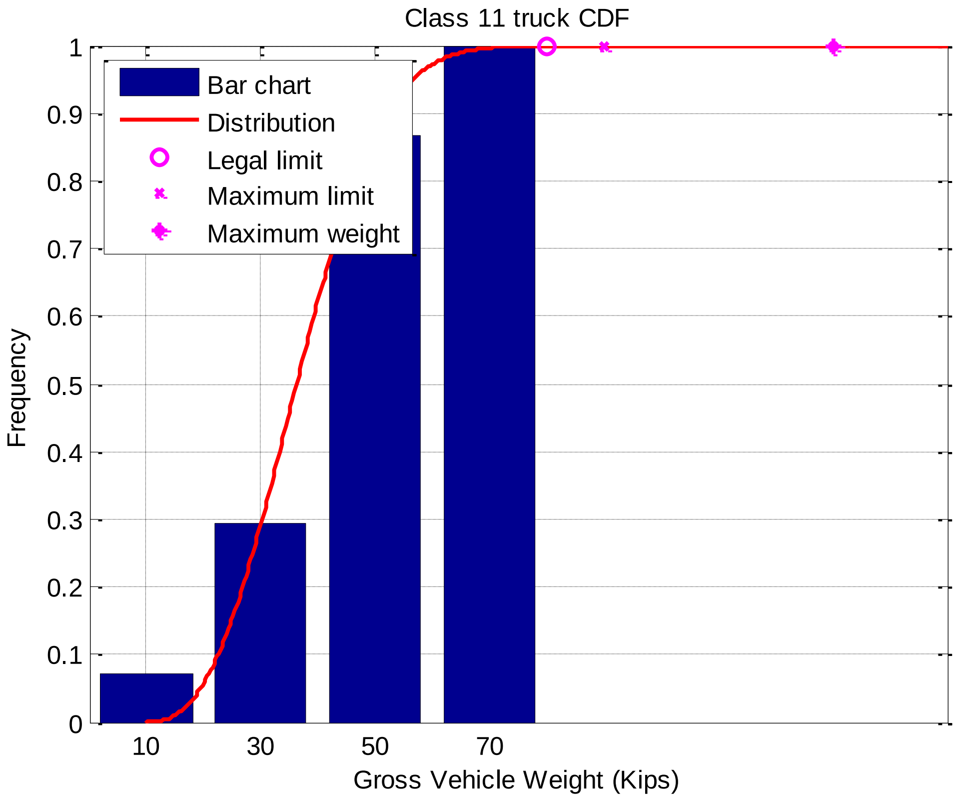


Figure A.17: Class 11 Truck CDF.

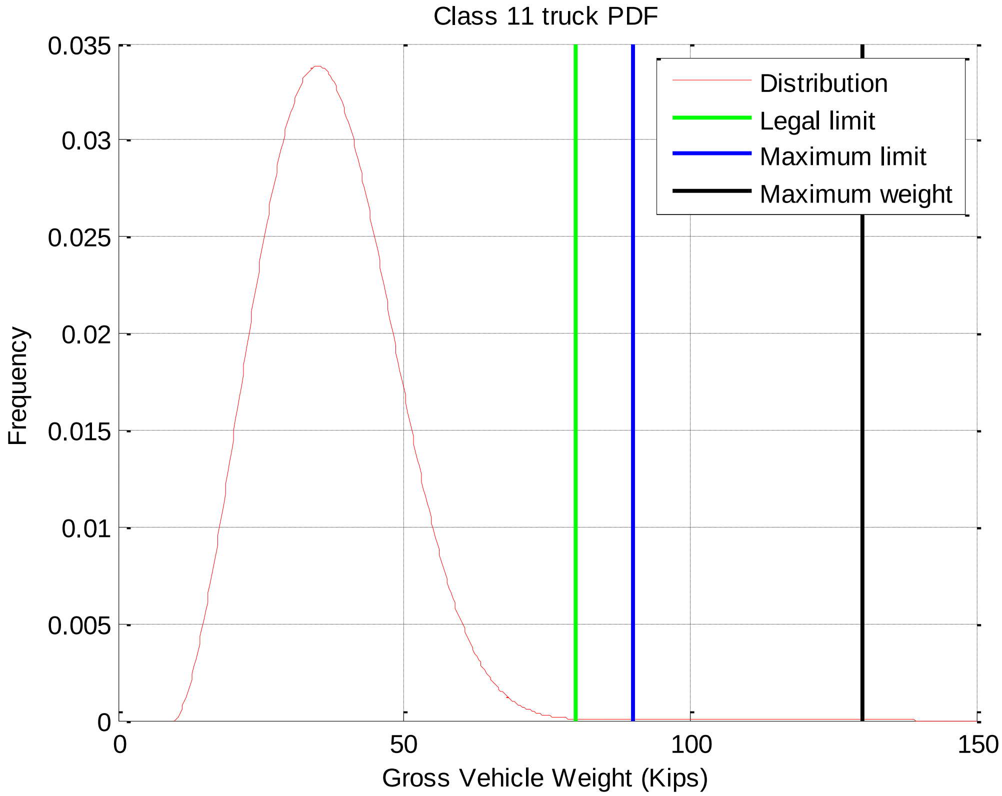


Figure A.18: Class 11 Truck PDF.

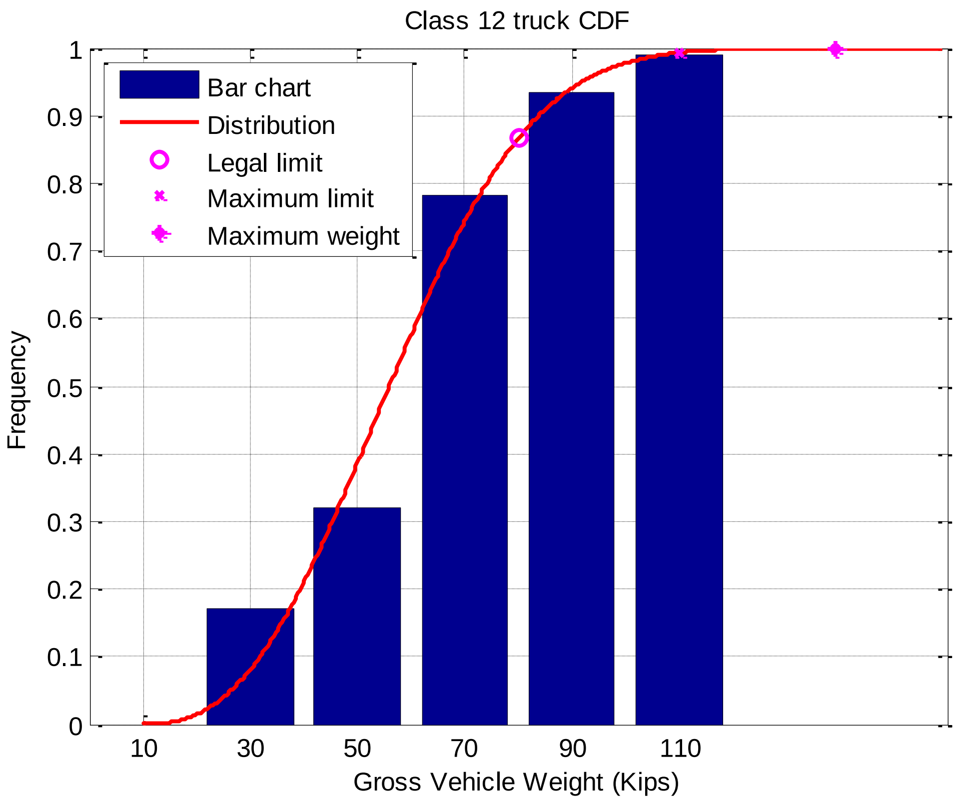


Figure A.19: Class 12 Truck CDF.

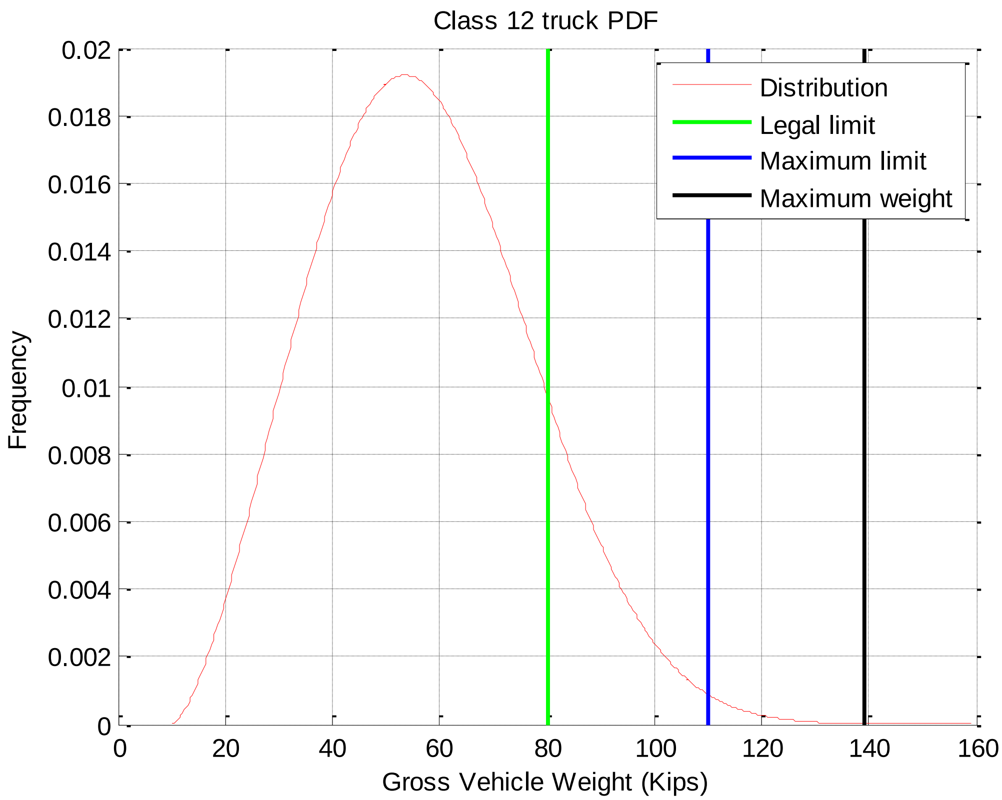


Figure A.20: Class 12 Truck PDF.

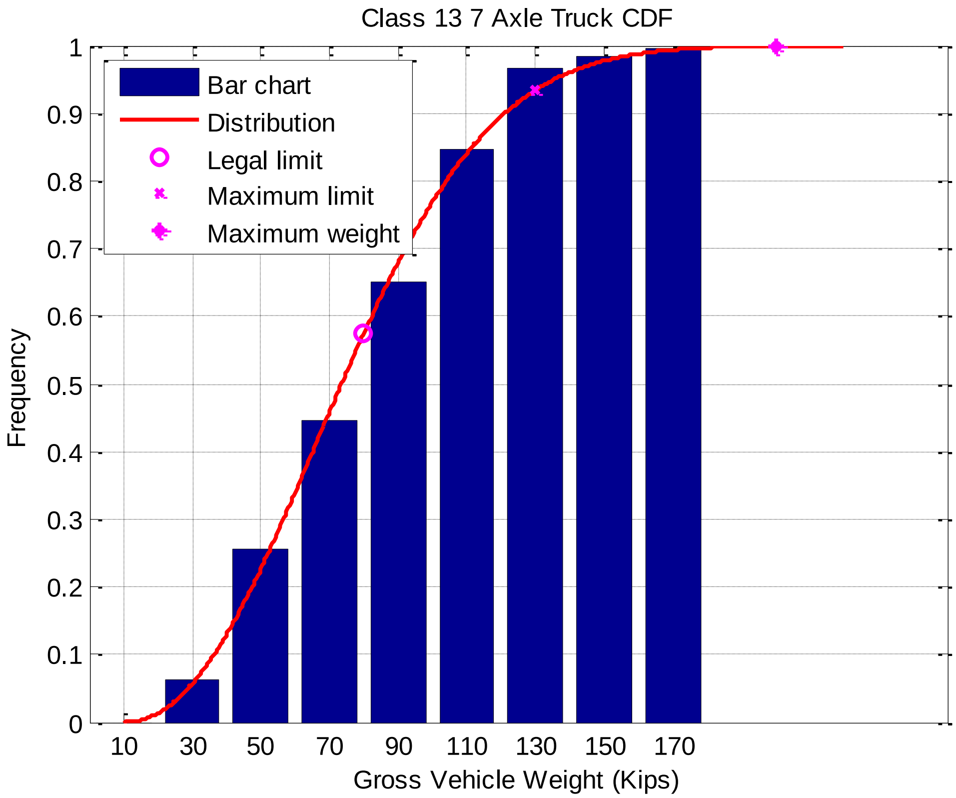


Figure A.21: Class 13 7 Axle Truck CDF.

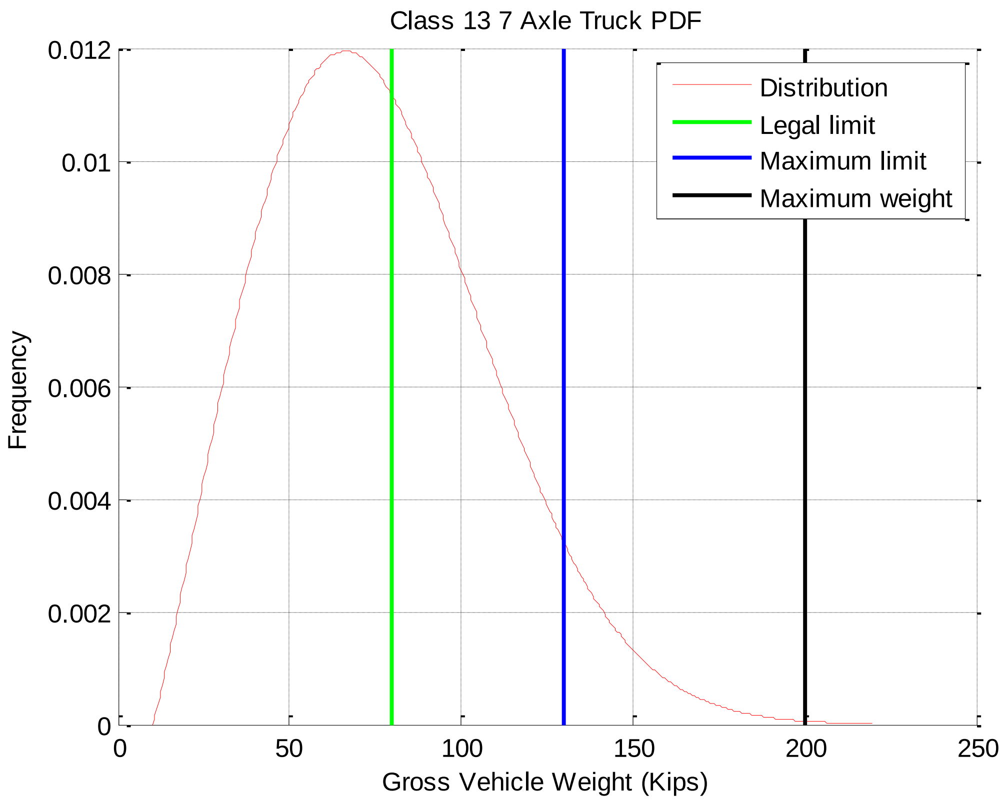


Figure A.22: Class 13 7 Axle Truck PDF.

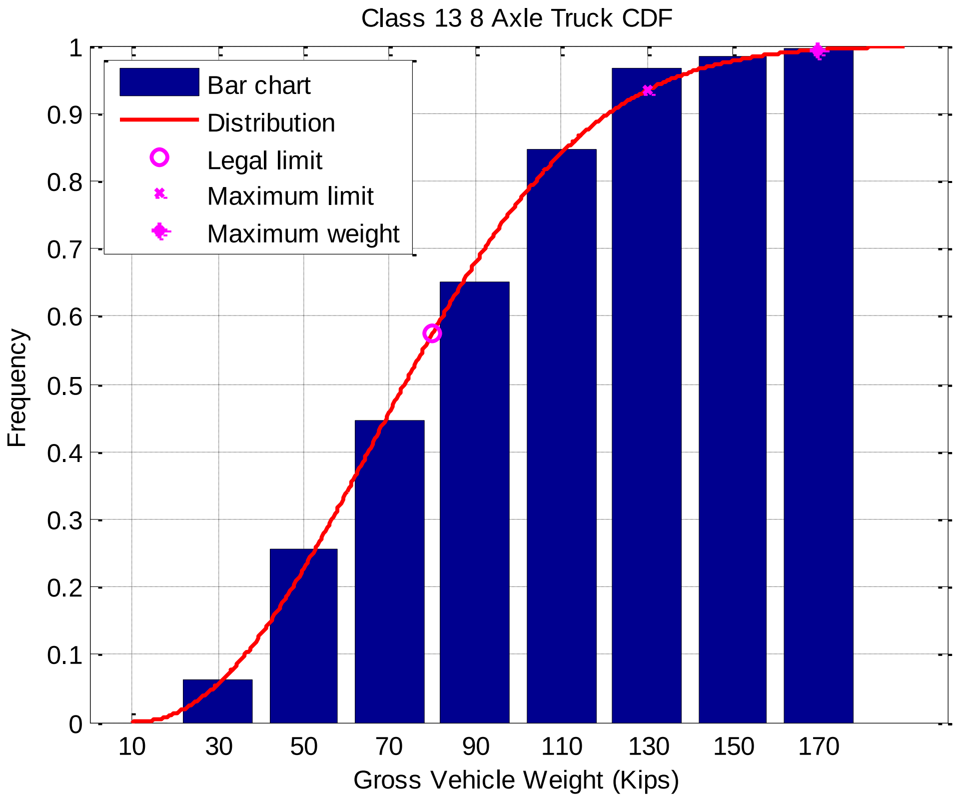


Figure A.23: Class 13 8 Axle Truck CDF.

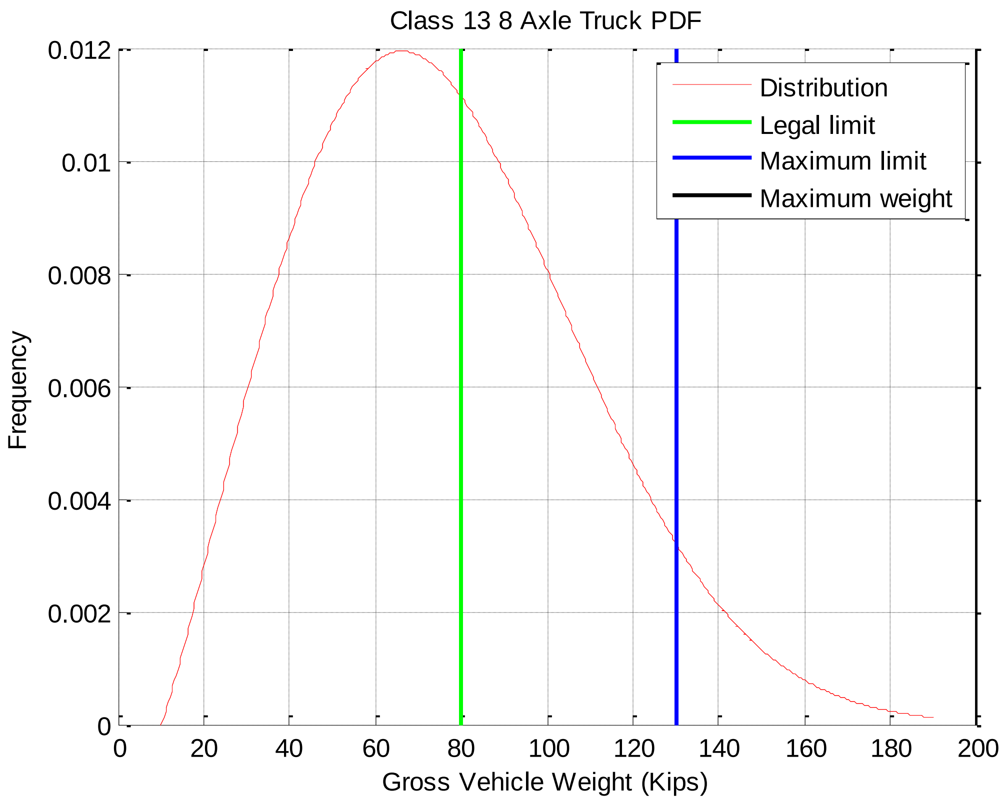


Figure A.24: Class 13 8 Axle Truck PDF.

Appendix B

SCDOT Overweight Trucks Permit Database

SCDOT overweight permit database was obtained from Oversize/Overweight Permit (OSOW) office, Sep 2012 (SCDOT 2012b).

2-Axle Trucks

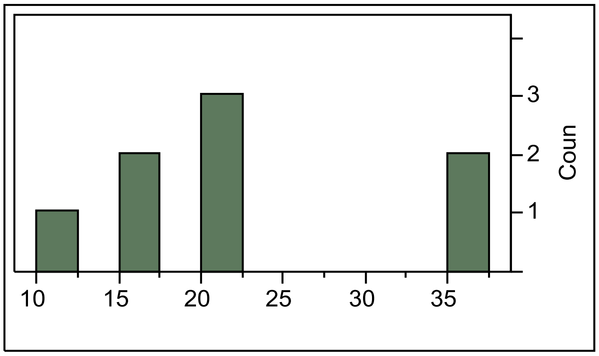


Figure B.1: 2-Axle Truck Spacing Configuration.

3-Axle Trucks

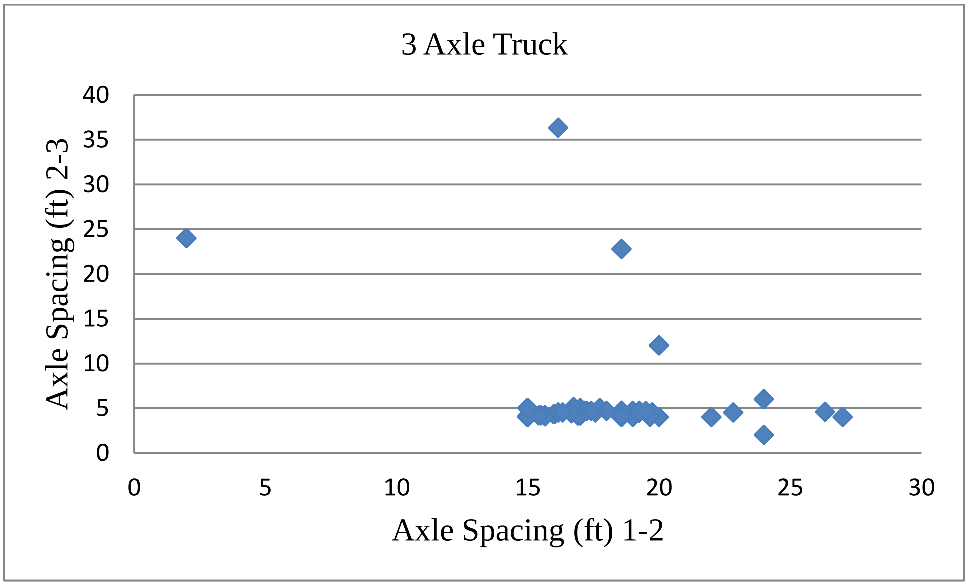


Figure B.2: 3-Axle Truck Spacing Configuration.

4-Axle Type A Trucks

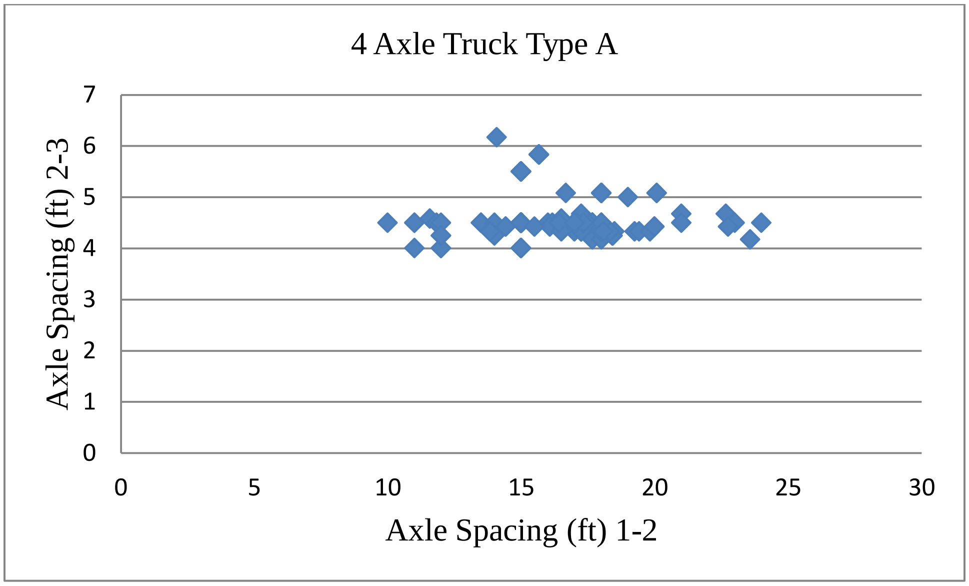


Figure B.3: 4-Axle Type A Truck Spacing Configuration 1.

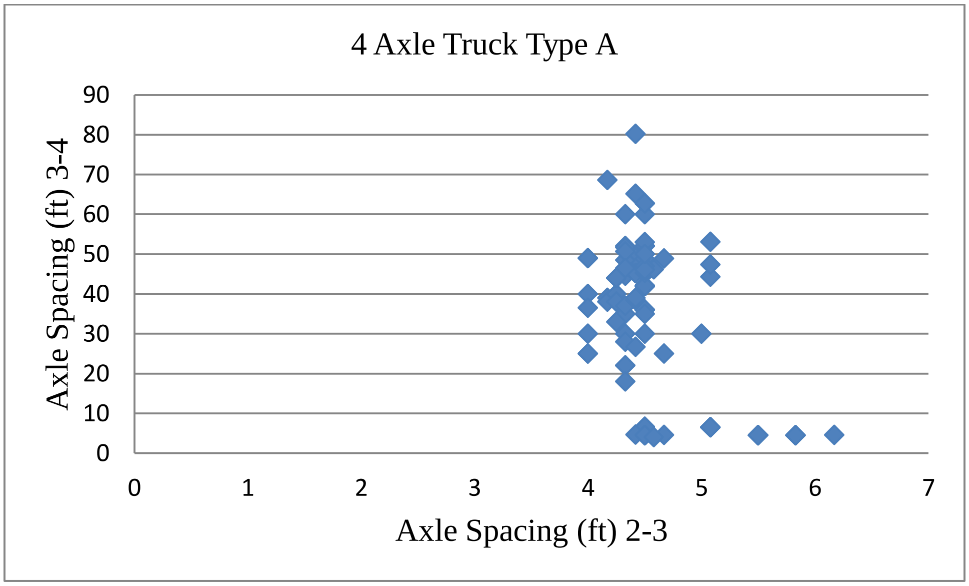


Figure B.4: 4-Axle Type A Truck Spacing Configuration 2.

4-Axle Type B Trucks

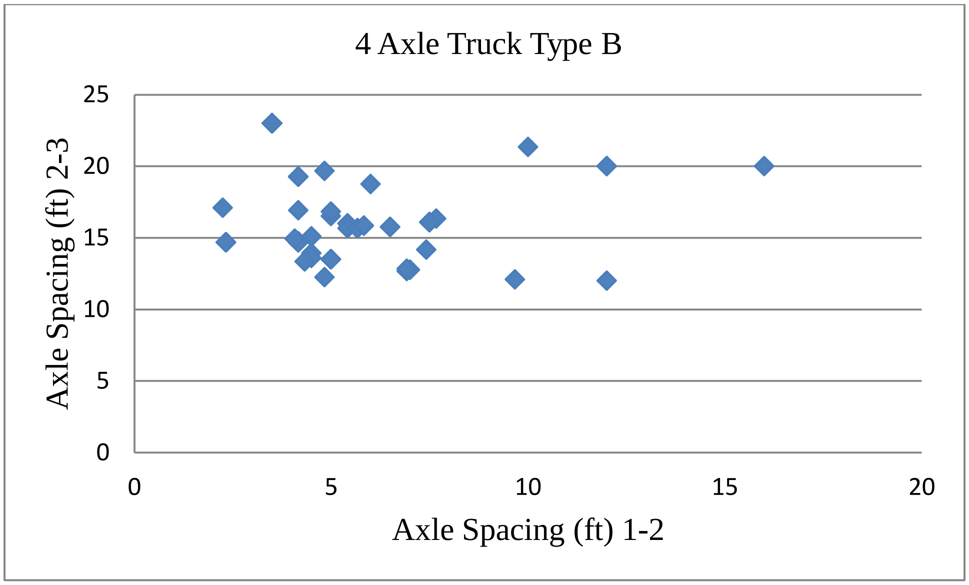


Figure B.5: 4-Axle Type B Truck Spacing Configuration 1.

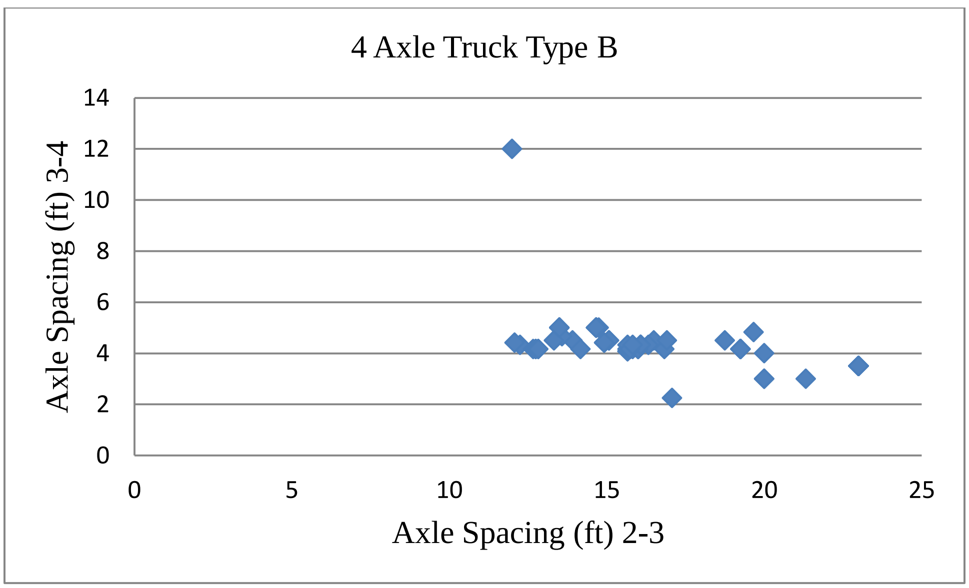


Figure B.6: 4-Axle Type B Truck Spacing Configuration 2.

4-Axle Type C Trucks

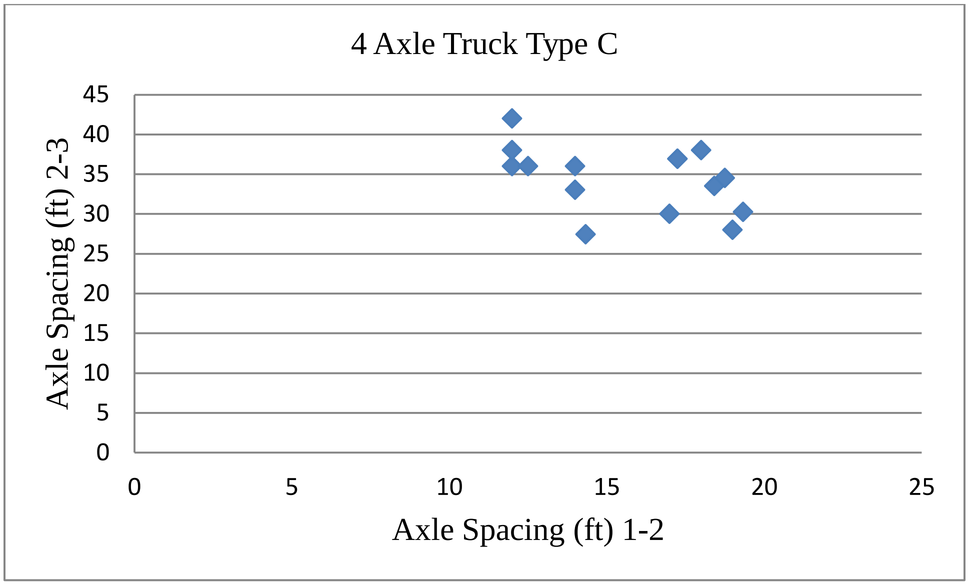


Figure B.7: 4-Axle Type C Truck Spacing Configuration 1.

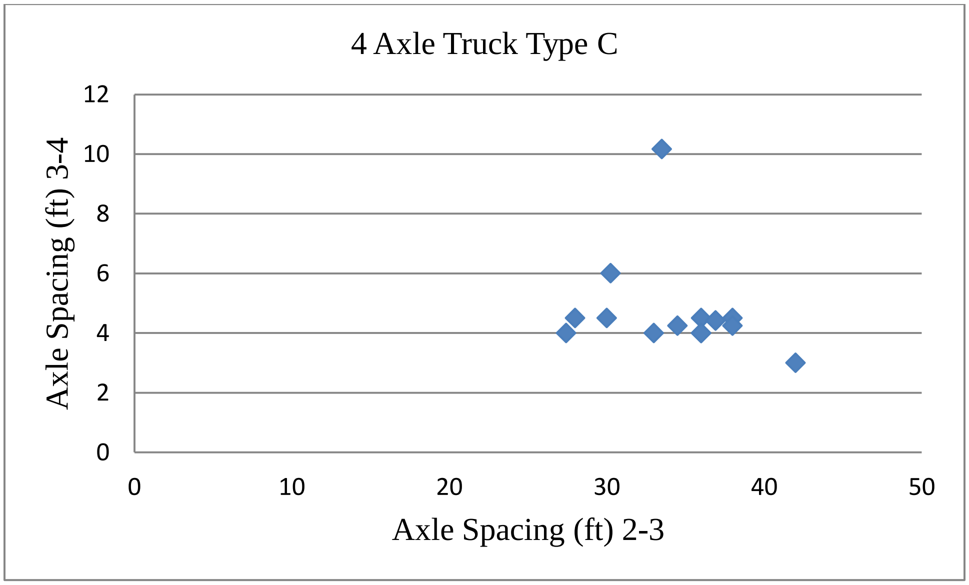


Figure B.8: 4-Axle Type C Truck Spacing Configuration 2.

5-Axle Trucks

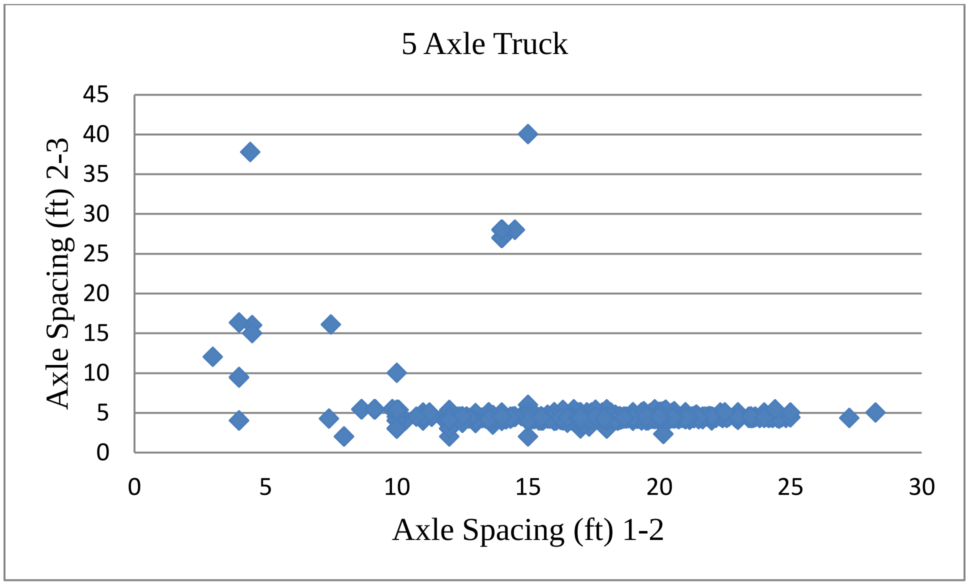


Figure B.9: 5-Axle Truck Spacing Configuration 1.

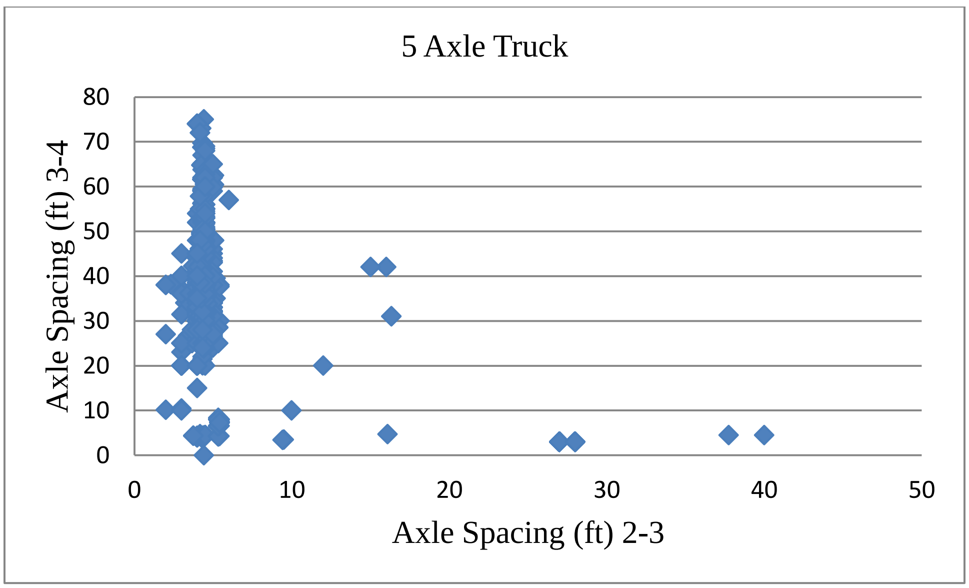


Figure B.10: 5-Axle Truck Spacing Configuration 2.

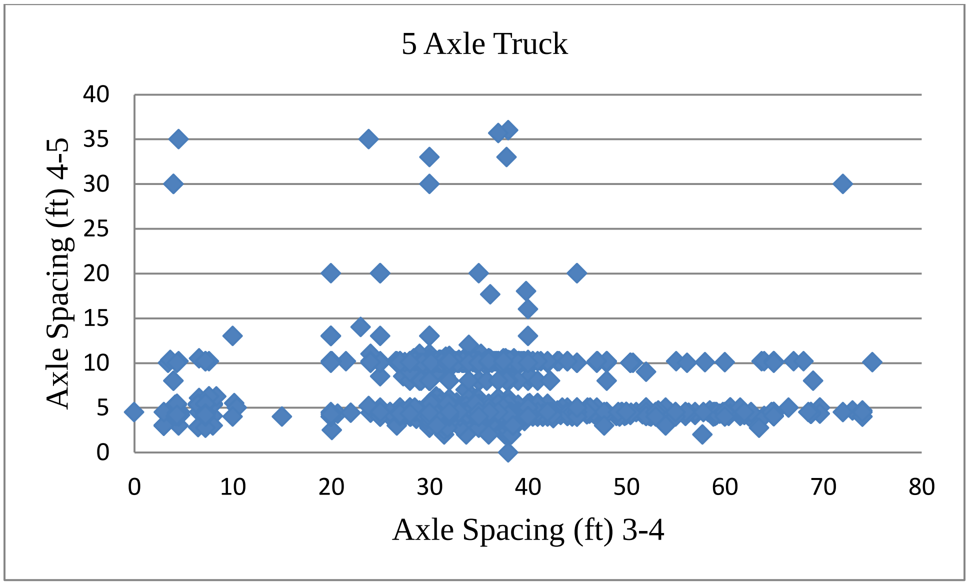


Figure B.11: 5-Axle Truck Spacing Configuration 3.

6-Axle Trucks

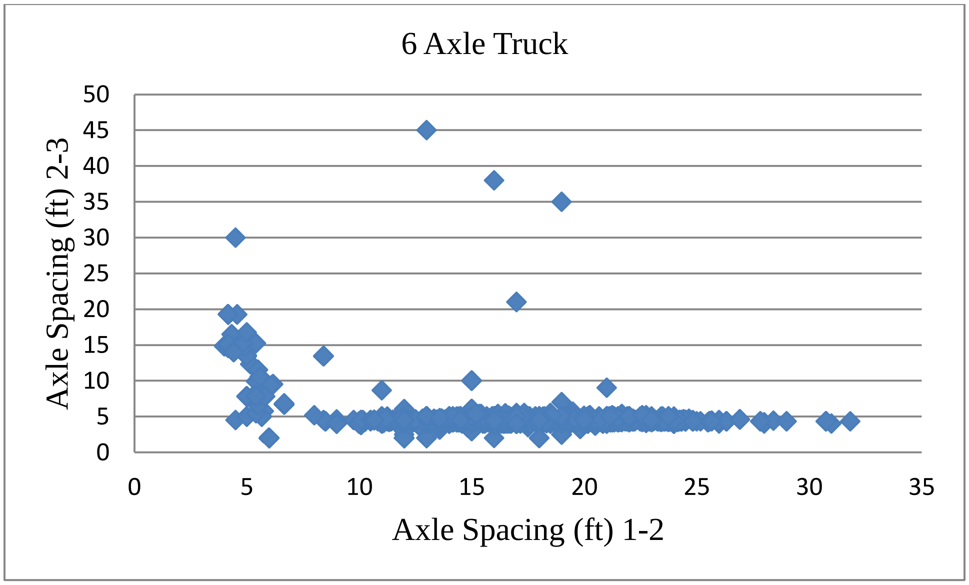


Figure B.12: 6-Axle Truck Spacing Configuration 1.

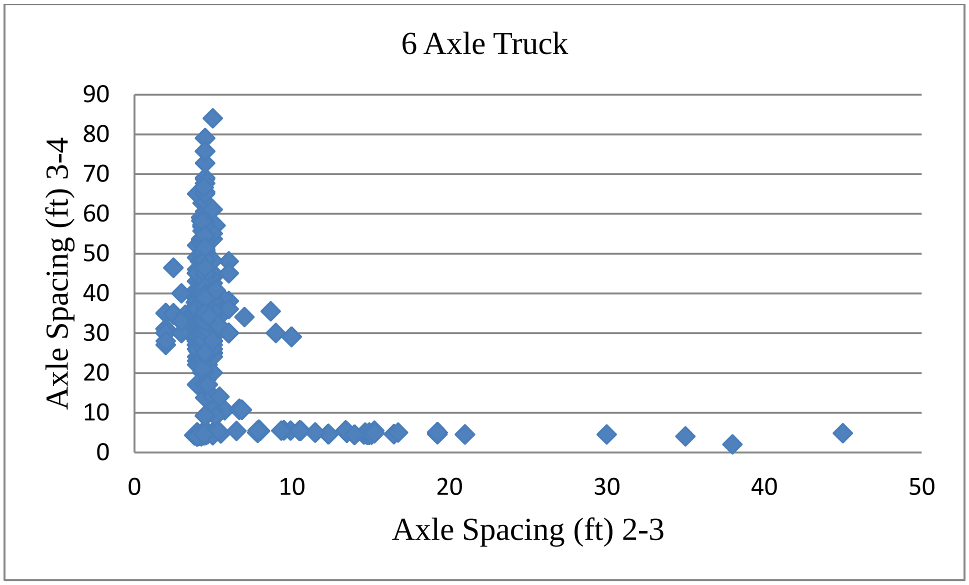


Figure B.13: 6-Axle Truck Spacing Configuration 2.

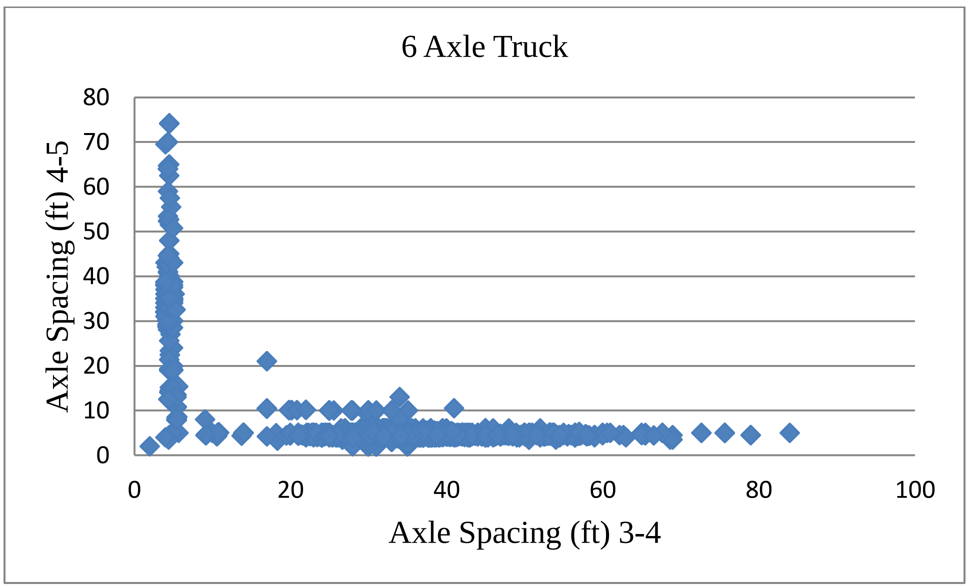


Figure B.14: 6-Axle Truck Spacing Configuration 3.

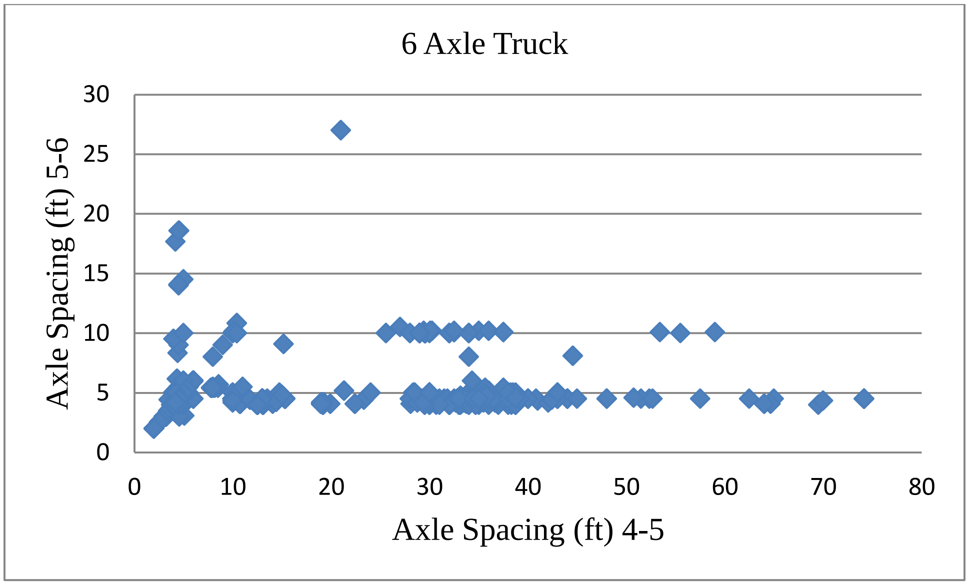


Figure B.15: 6-Axle Truck Spacing Configuration 4.

7-Axle Trucks

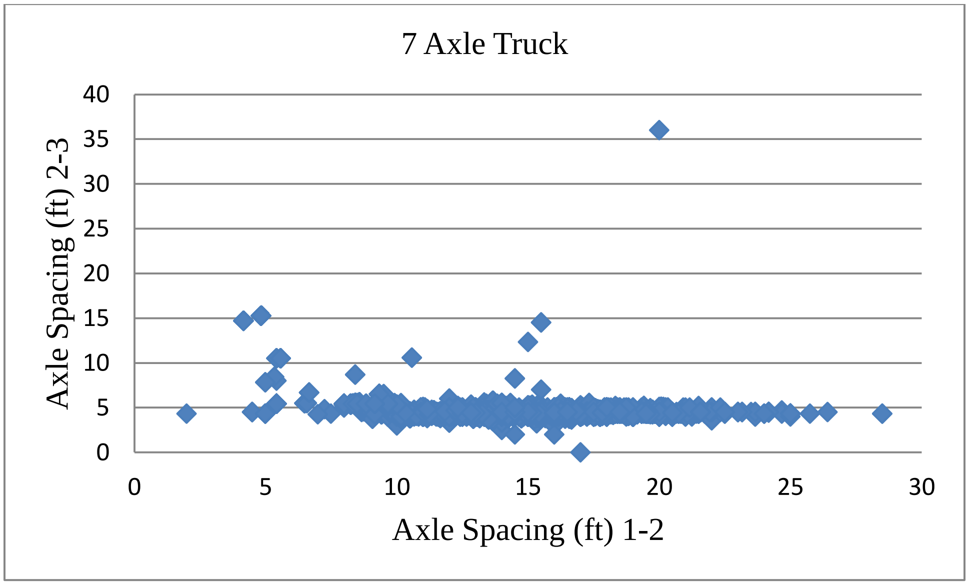


Figure B.16: 7-Axle Truck Spacing Configuration 1.

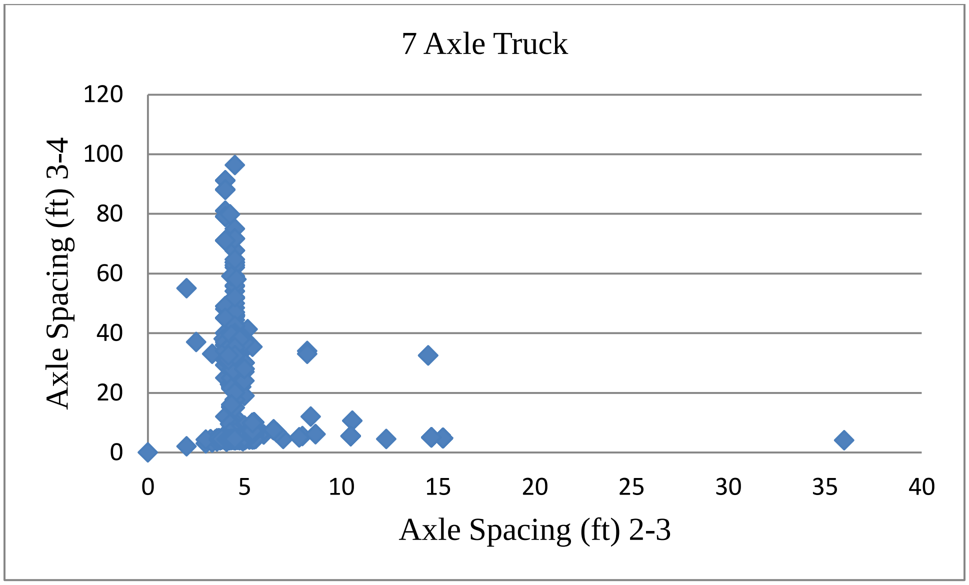


Figure B.17: 7-Axle Truck Spacing Configuration 2.

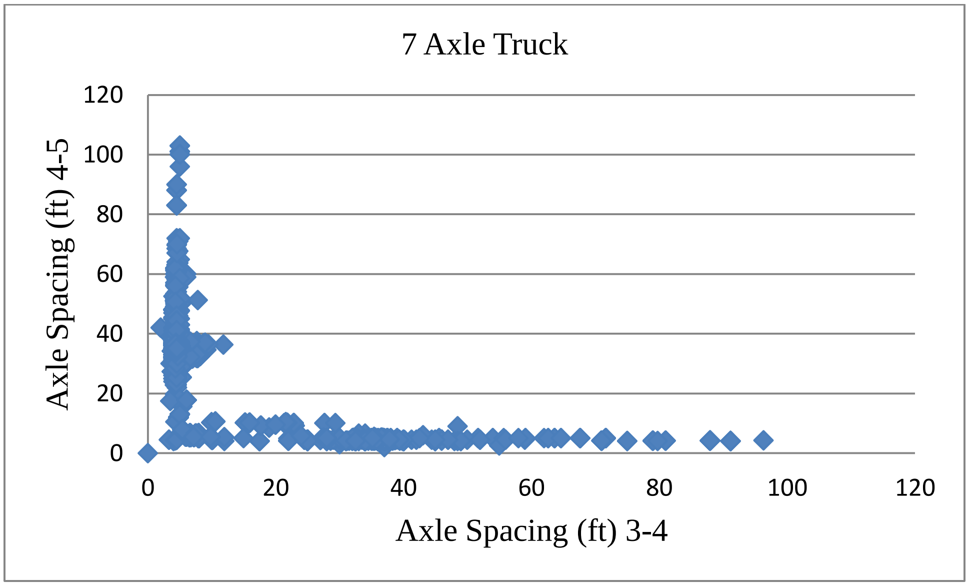


Figure B.18: 7-Axle Truck Spacing Configuration 3.

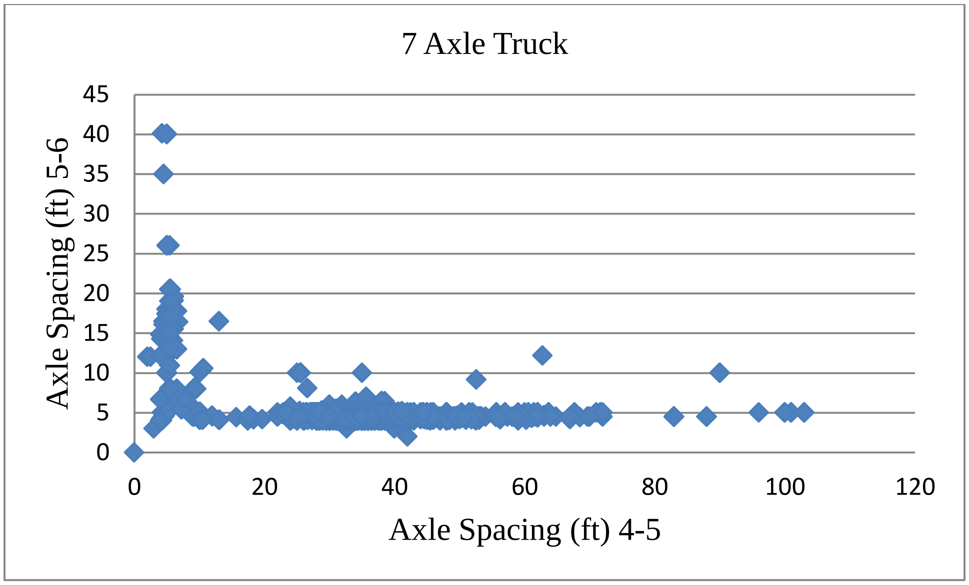


Figure B.19: 7-Axle Truck Spacing Configuration 4.

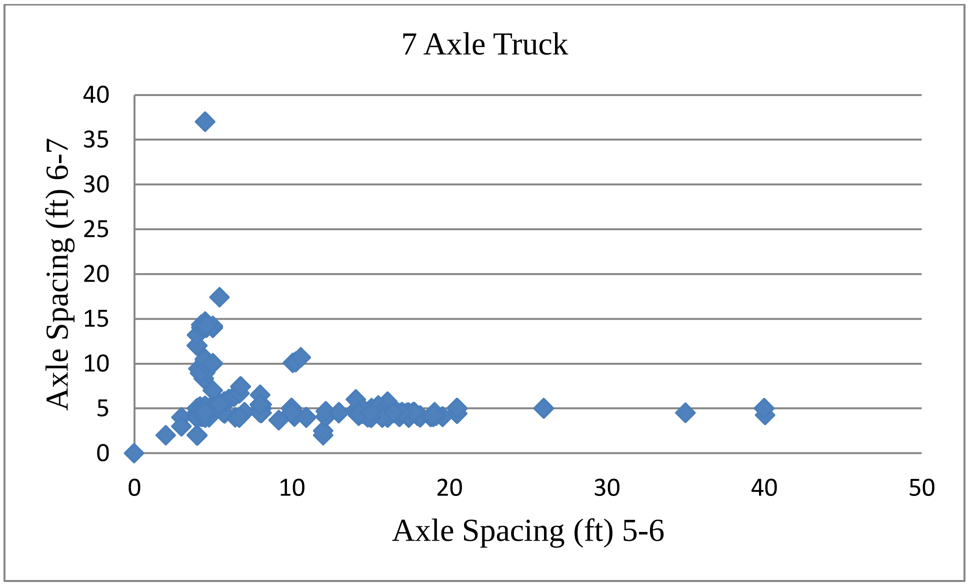


Figure B.20: 7-Axle Truck Spacing Configuration 5.

<u>8-Axle Trucks</u>

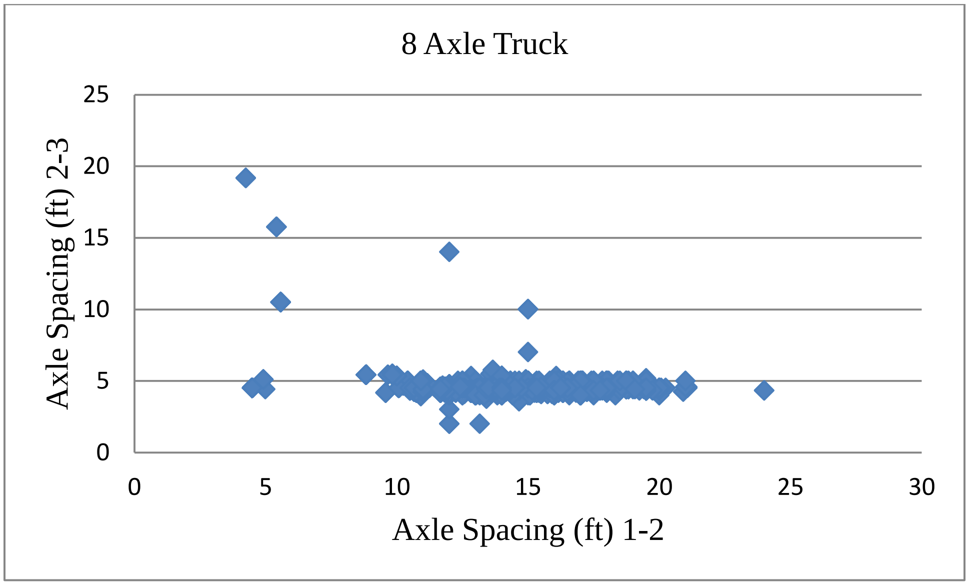


Figure B.21: 8-Axle Truck Spacing Configuration 1.

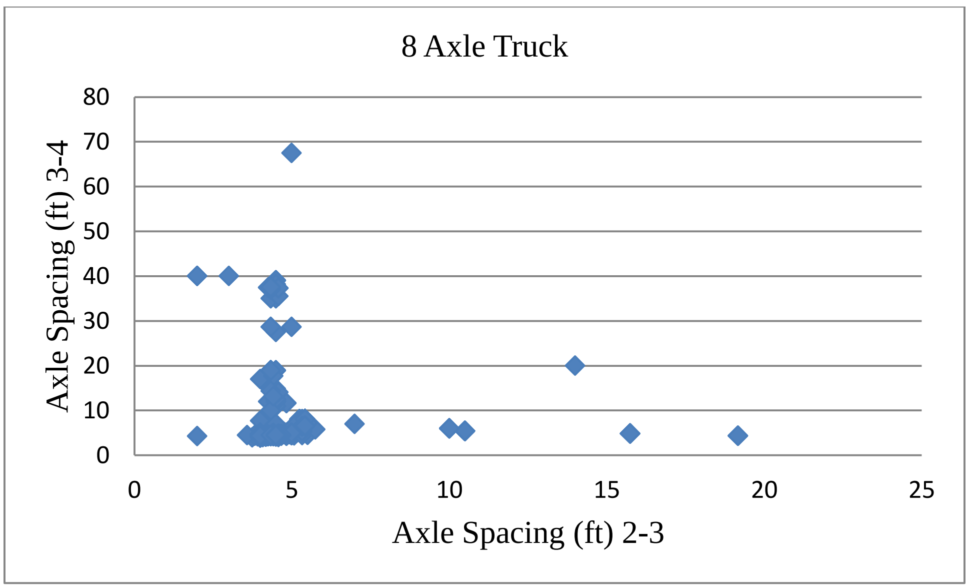


Figure B.22: 8-Axle Truck Spacing Configuration 2.

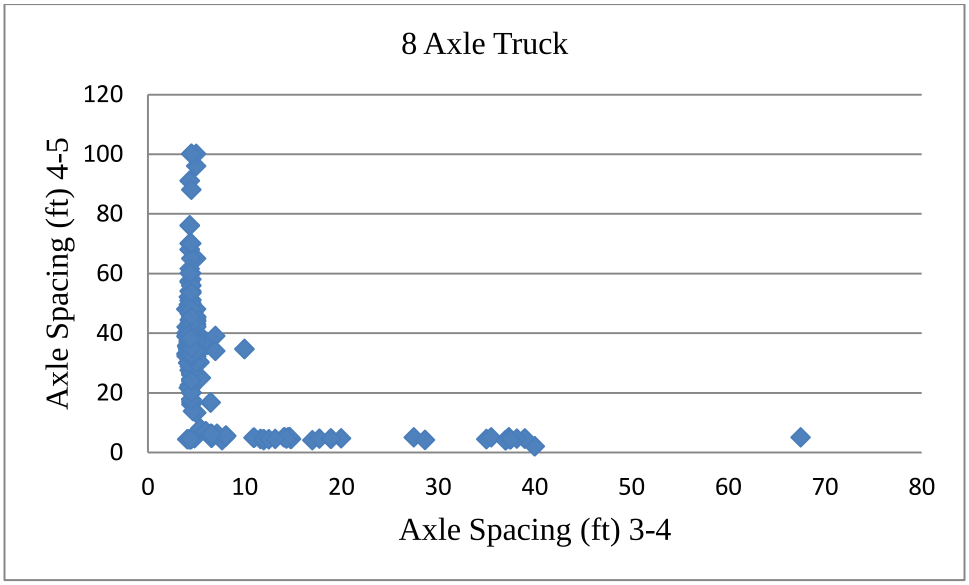


Figure B.23: 8-Axle Truck Spacing Configuration 3.

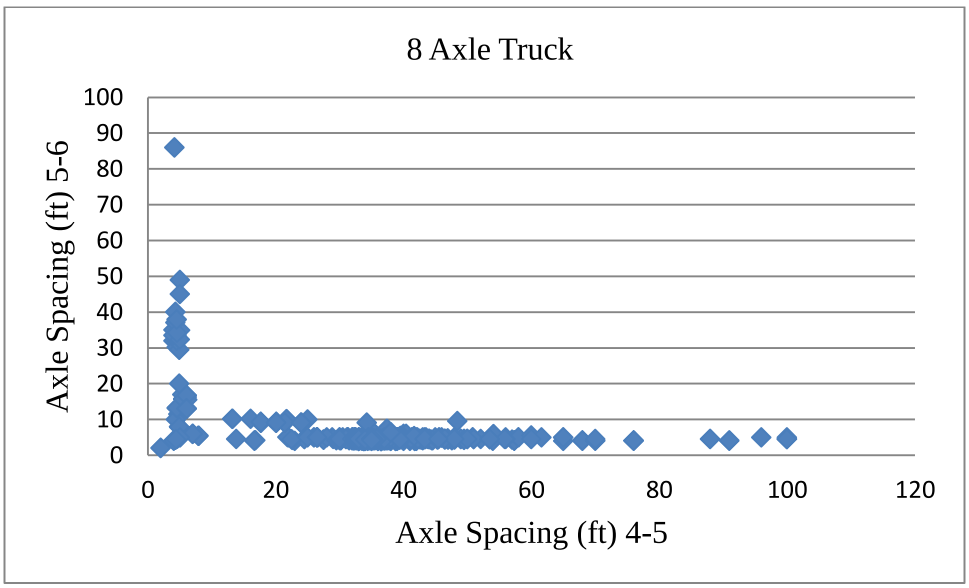


Figure B.24: 8-Axle Truck Spacing Configuration 4.

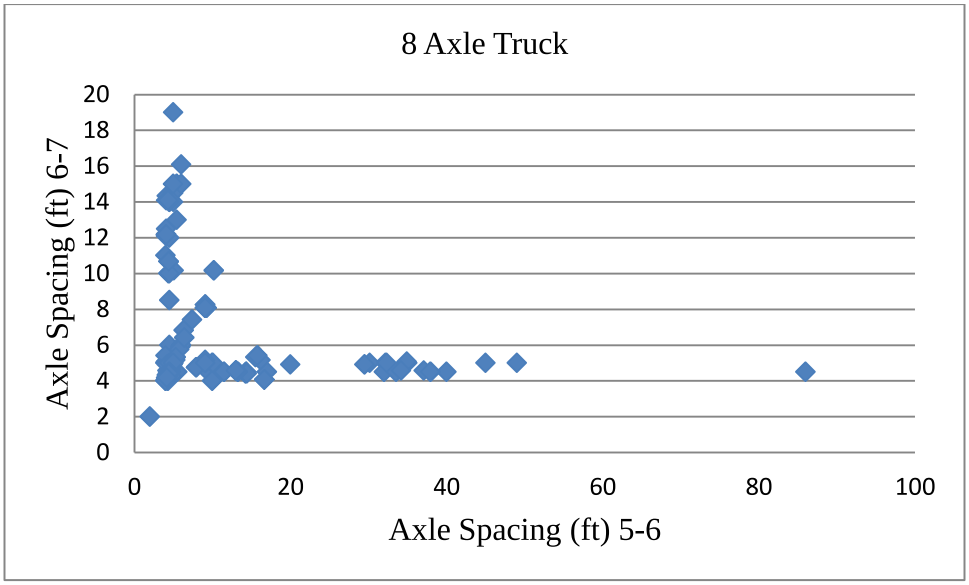


Figure B.25: 8-Axle Truck Spacing Configuration 5.

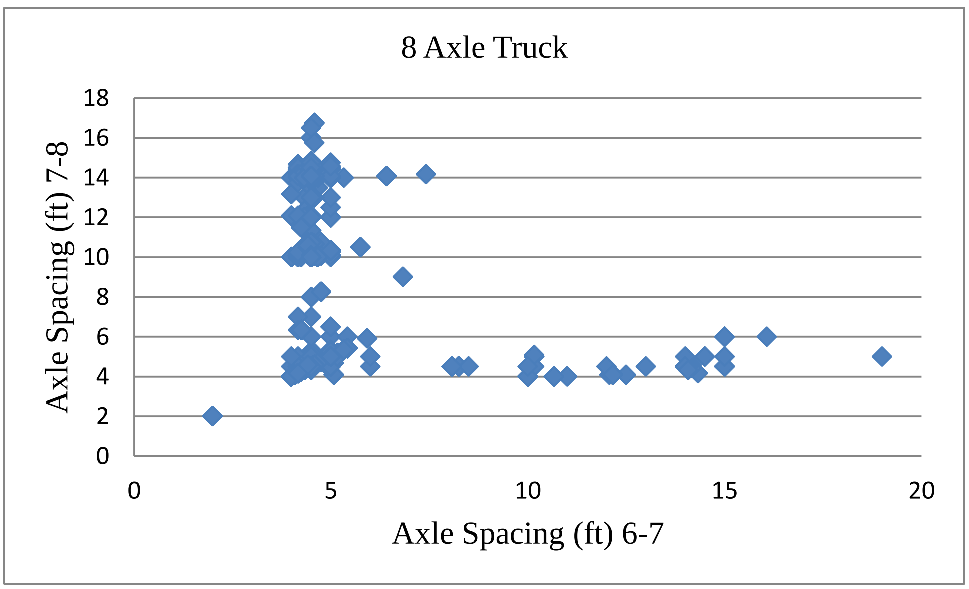


Figure B.26: 8-Axle Truck Spacing Configuration 6.

Appendix C

Bridge Replacement Cost Models

Bridge Replacement Cost Models Development

Table C.1: Bridge Cost Models Parameters.

Cost Model Number	a_1[(a)]	b_1[(a)]	RMS_1[(a)]	a_2[(a)]	b_2[(a)]	RMS_2[(a)]	Average Unit Area Cost (x$1000/$m^2$) [(a)]
1	2.649	1.445	413.5	1.944	0.990	492.6	1.422
2	75.307	0.688	16.8	0.835	1.071	5.0	1.338
4	22.128	0.926	81.0	1.856	0.966	49.1	1.549
7	67.174	0.580	3.2	0.583	1.159	0.8	1.428
9	29.225	0.868	36.3	9.380	0.638	35.4	1.679
12	9.814	1.080	53.1	4.219	0.798	69.7	1.002
13	48.238	0.608	3.9	41.035	0.352	4.0	0.694
14	0.000	0.000	0.0	0.000	0.000	0.0	1.323
16	1068.053	0.171	31.0	1033.993	0.126	32.0	1.943
18	0.930	1.490	300.7	1.559	0.989	118.7	1.532
19	0.000	0.000	0.0	0.000	0.000	0.0	1.554
21	5.879	1.078	1.2	1.286	1.013	3.6	1.425
22	0.000	0.000	0.0	0.000	0.000	0.0	1.585
23	0.000	0.000	0.0	0.000	0.000	0.0	1.565
24	0.000	0.000	0.0	0.000	0.000	0.0	1.446
25	0.000	0.000	0.0	0.000	0.000	0.0	2.295
27	0.000	0.000	0.0	0.000	0.000	0.0	0.833
28	65.277	0.775	593.8	12.888	0.731	608.9	1.268

29	0.000	0.000	0.0	0.000	0.000	0.0	1.594
30	0.000	0.000	0.0	0.000	0.000	0.0	1.427
31	0.000	0.000	0.0	0.000	0.000	0.0	1.116
32	0.000	0.000	0.0	0.000	0.000	0.0	0.560
34	8.050	1.095	28.8	1.232	0.979	28.0	1.139
35	18.699	0.961	188.5	3.451	0.870	198.4	1.496
38	3.567	1.311	3.6	0.377	1.159	2.6	0.870
41	0.000	0.000	0.0	0.000	0.000	0.0	0.805
42	6.034	1.158	299.8	0.002	1.617	318.8	1.209
43	0.000	0.000	0.0	0.000	0.000	0.0	1.428
44	0.000	0.000	0.0	0.000	0.000	0.0	1.791
45	10.674	0.942	1.8	4.100	0.790	1.0	1.813
46	0.000	0.000	0.0	0.000	0.000	0.0	1.501
47	0.000	0.000	0.0	0.000	0.000	0.0	1.451
48	0.000	0.000	0.0	0.000	0.000	0.0	2.573

(a) More details about cost model parameters are discussed in Chapter 7.

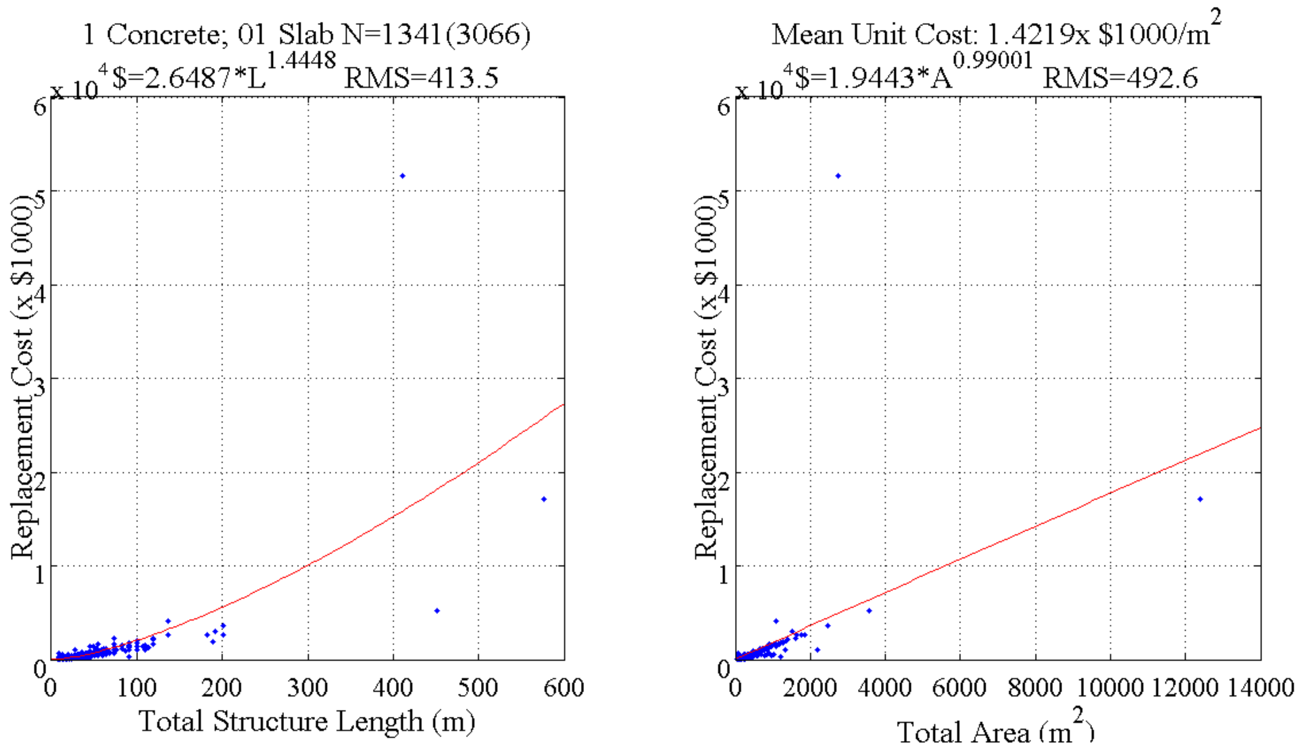


Figure C.1: Cost Model 1.

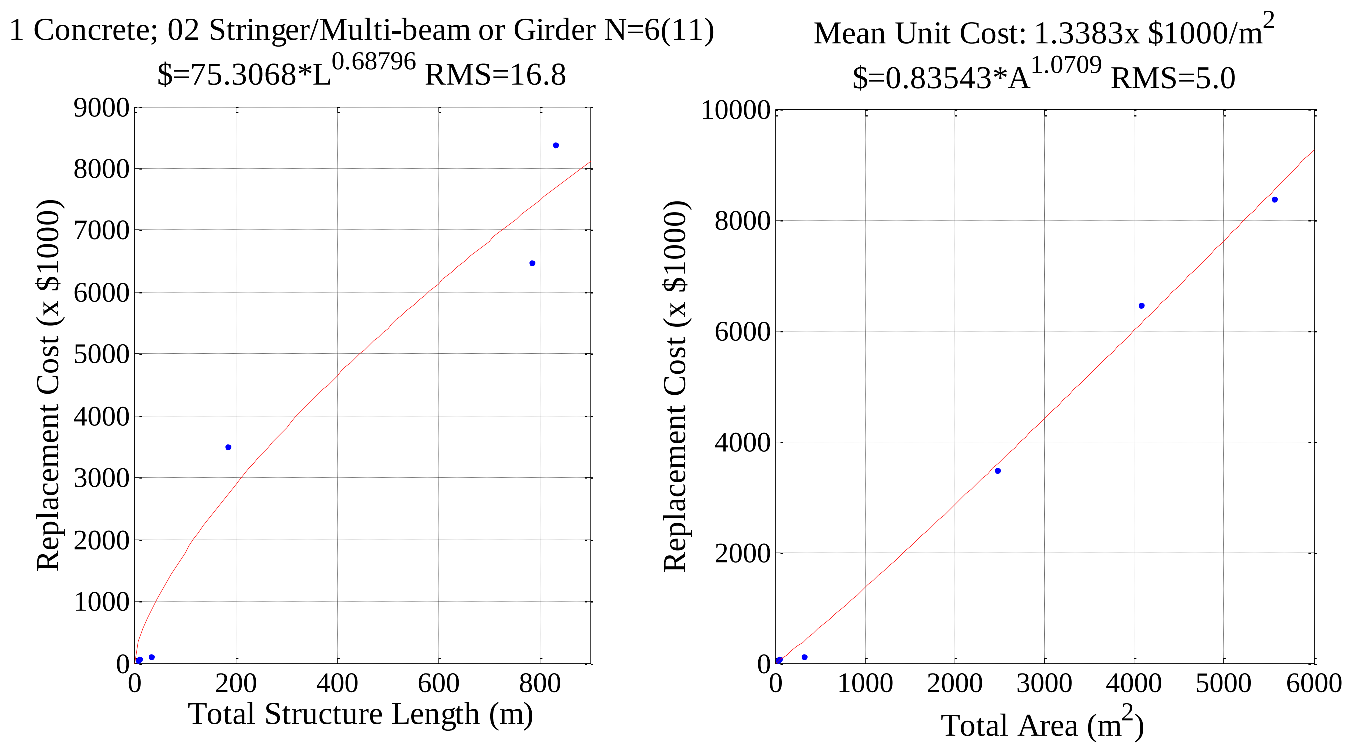


Figure C.2: Cost Model 2.

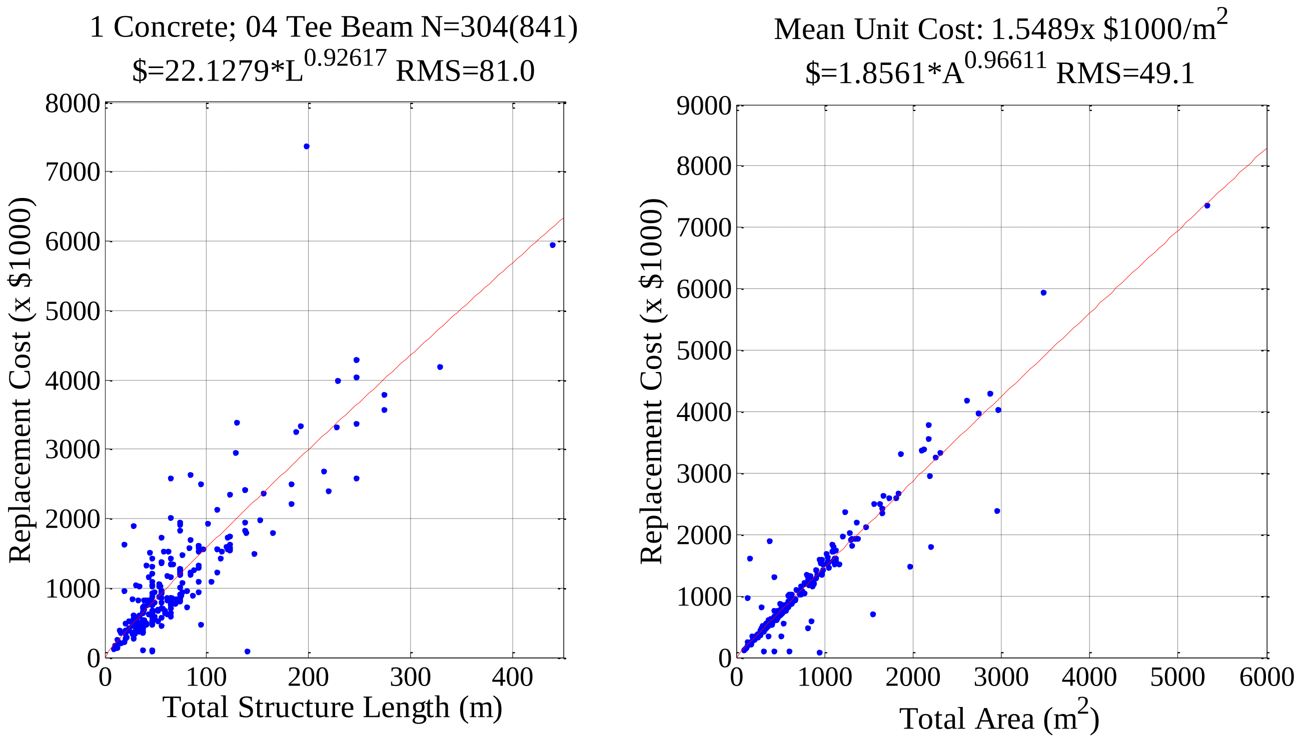


Figure C.3: Cost Model 4.

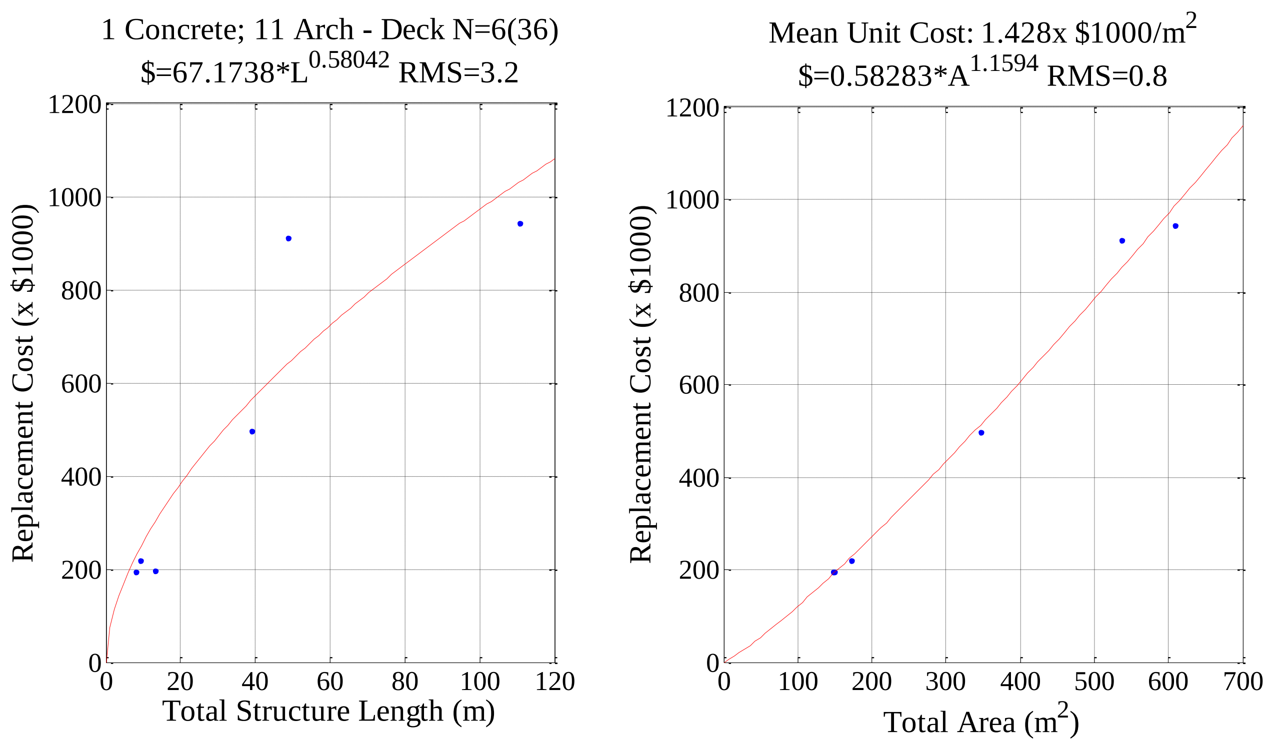


Figure C.4: Cost Model 7.

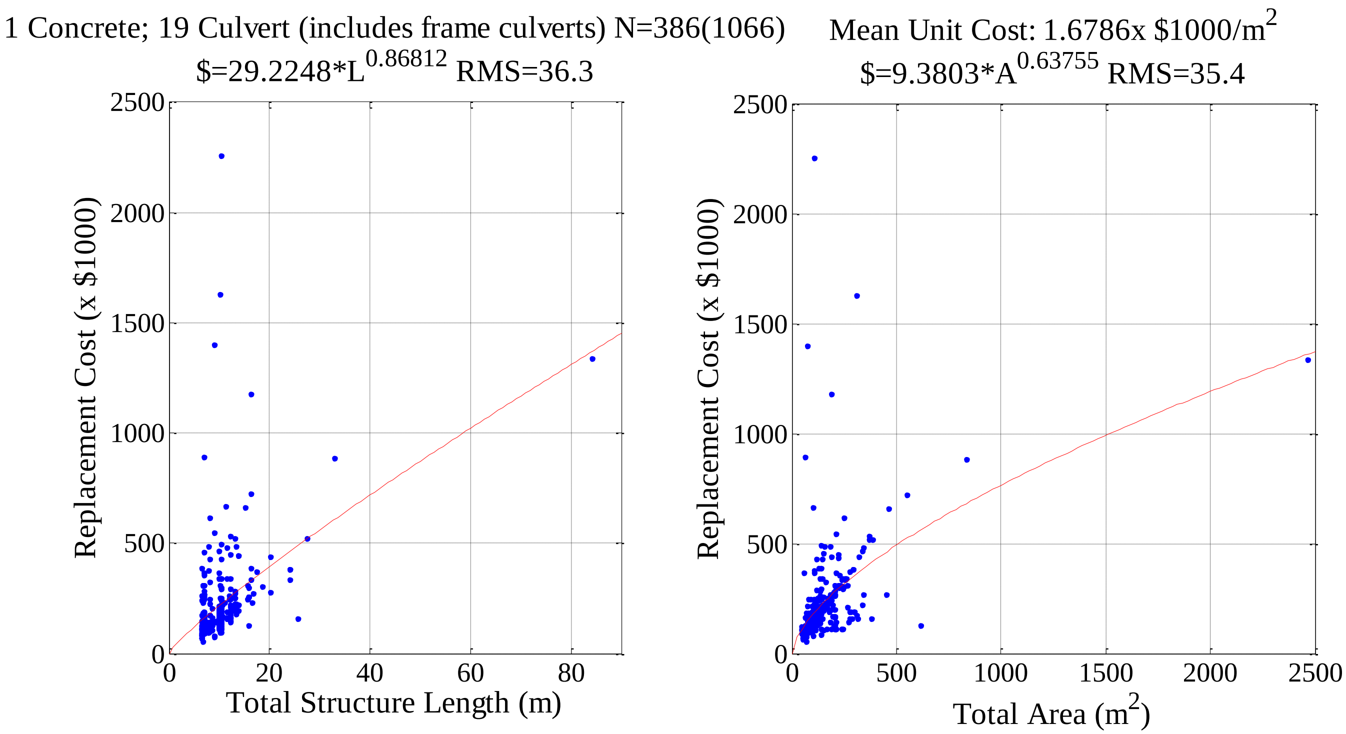


Figure C.5: Cost Model 9.

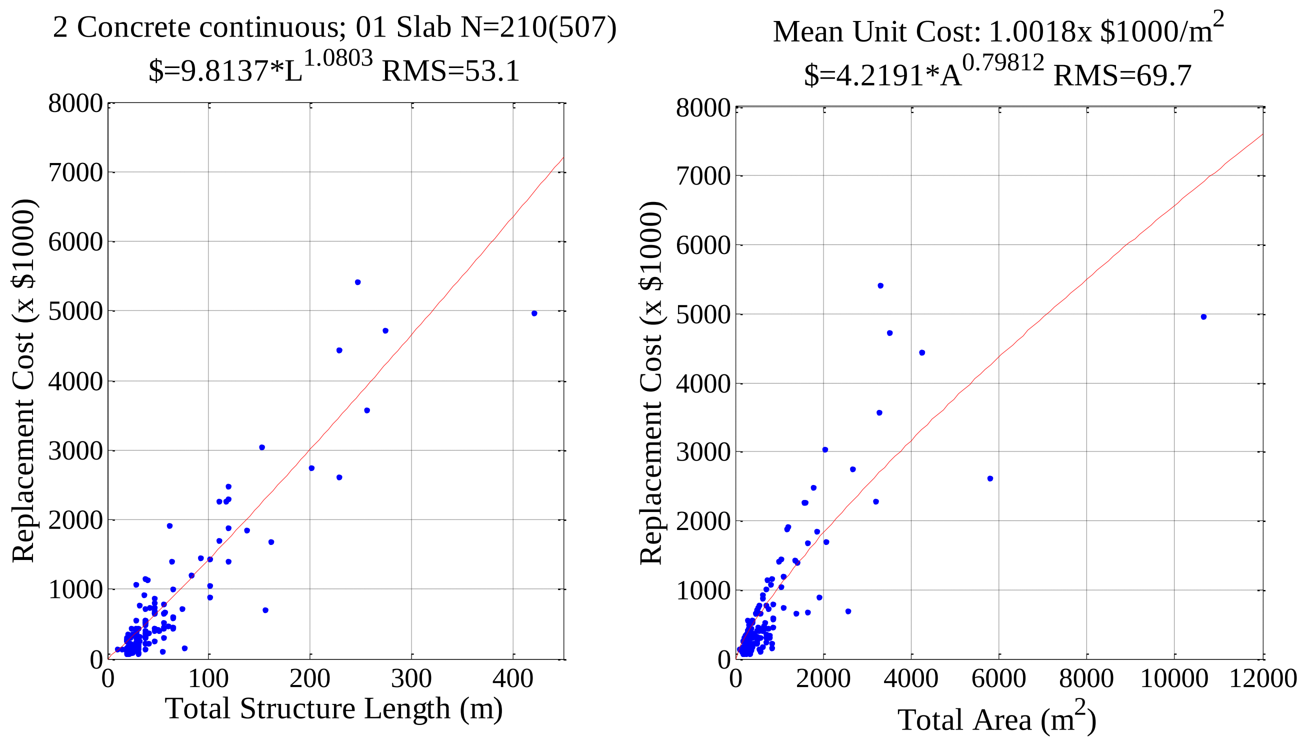


Figure C.6: Cost Model 12.

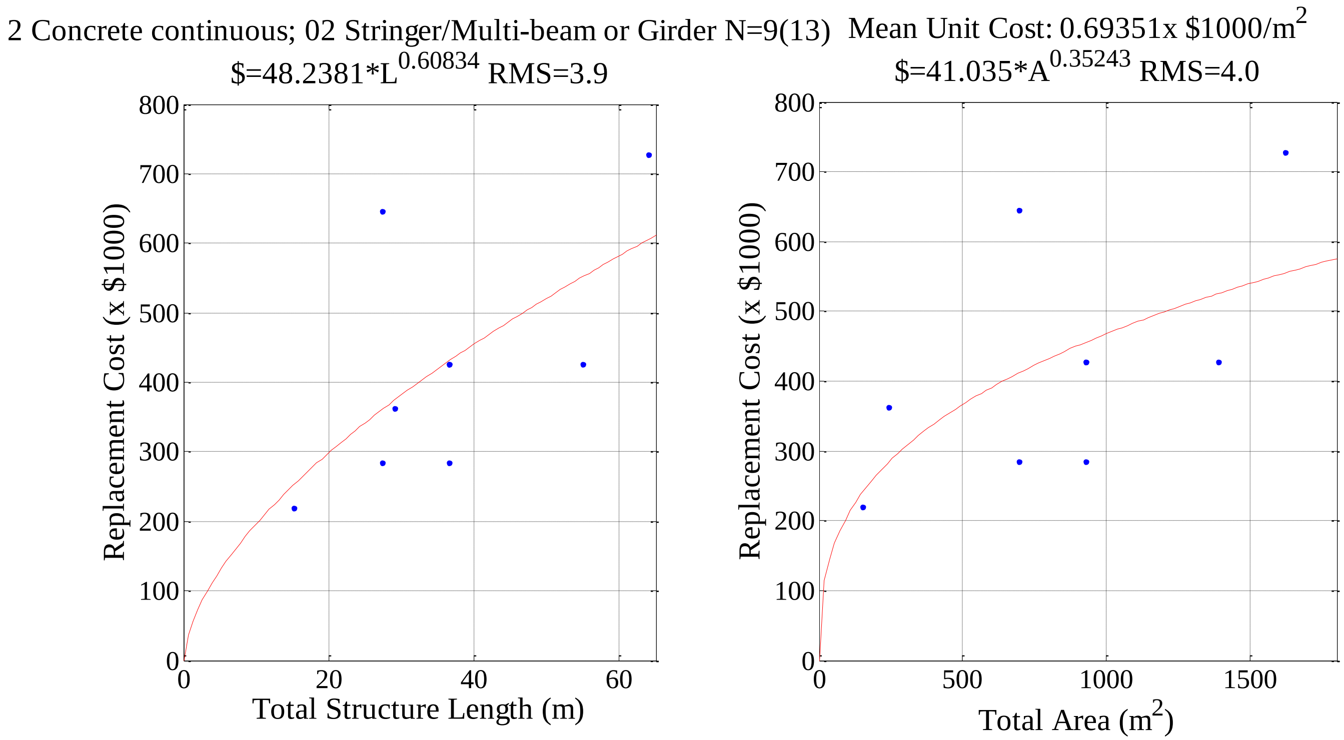


Figure C.7: Cost Model 13.

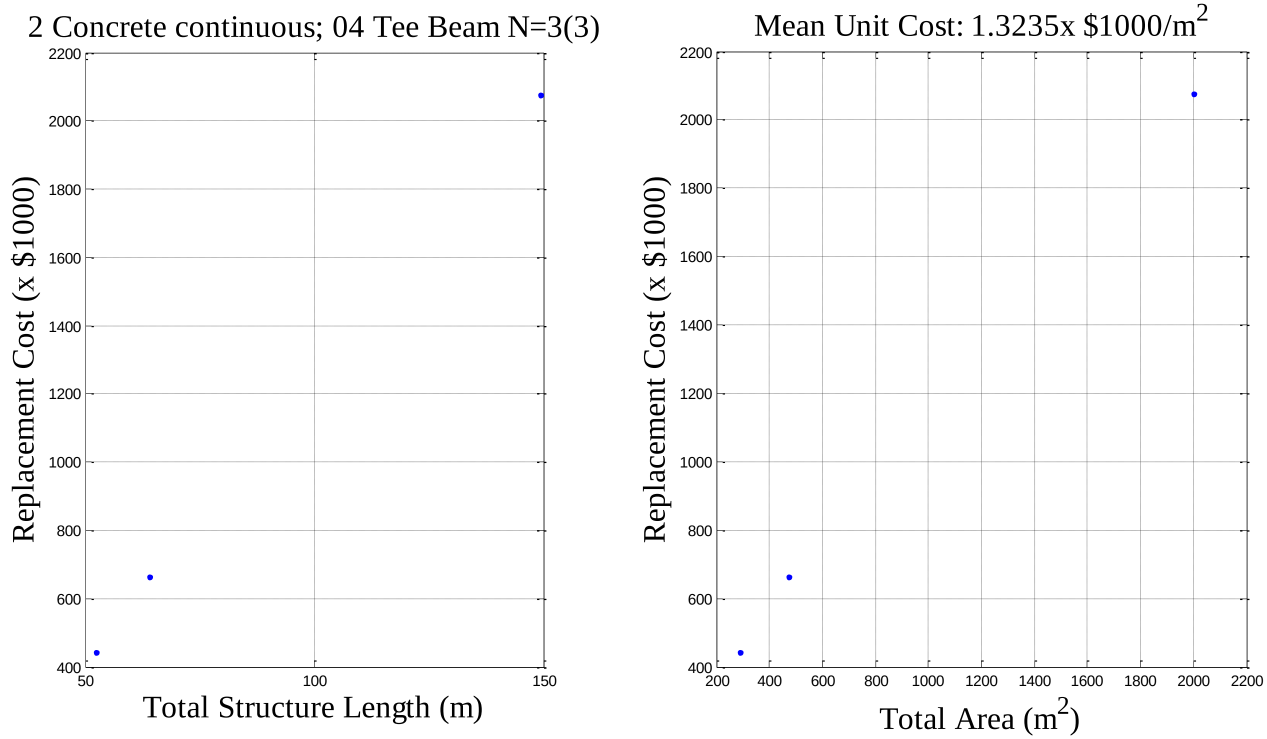


Figure C.8: Cost Model 14.

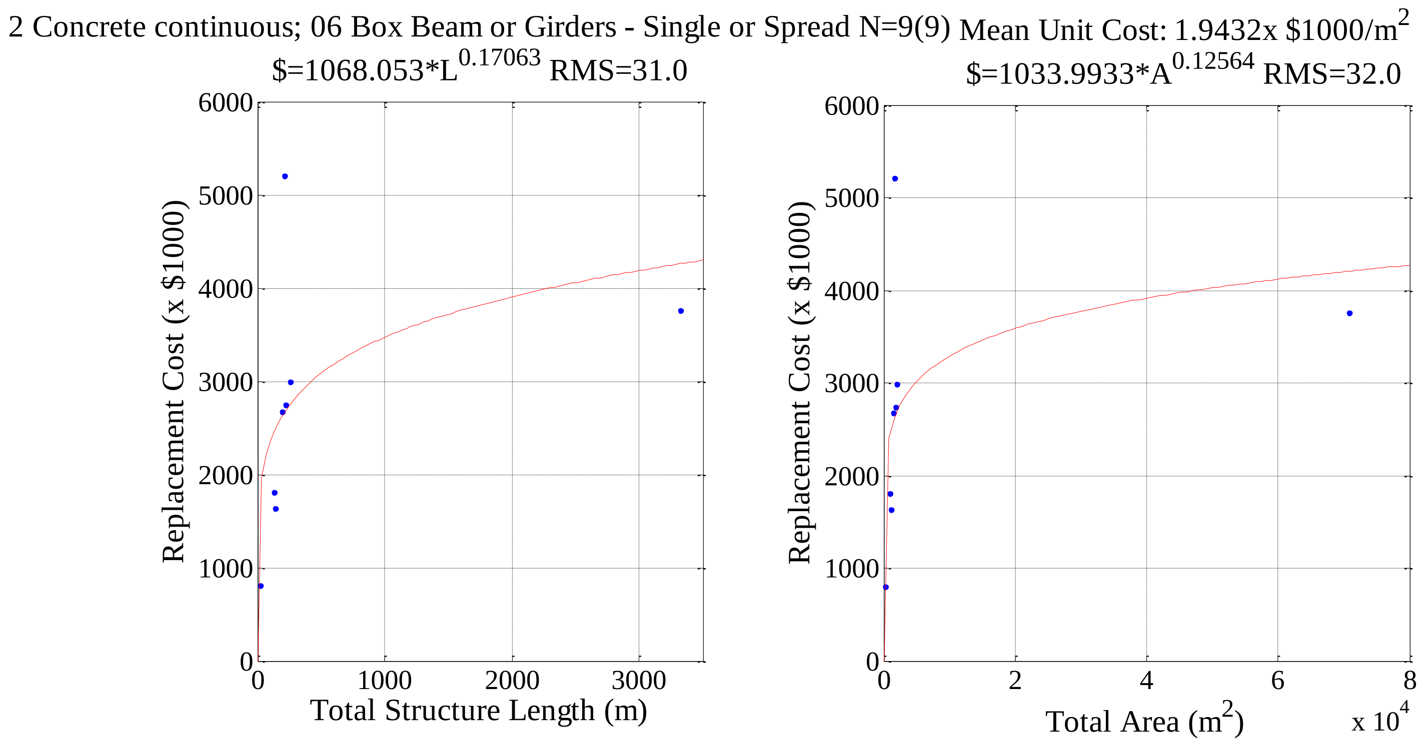
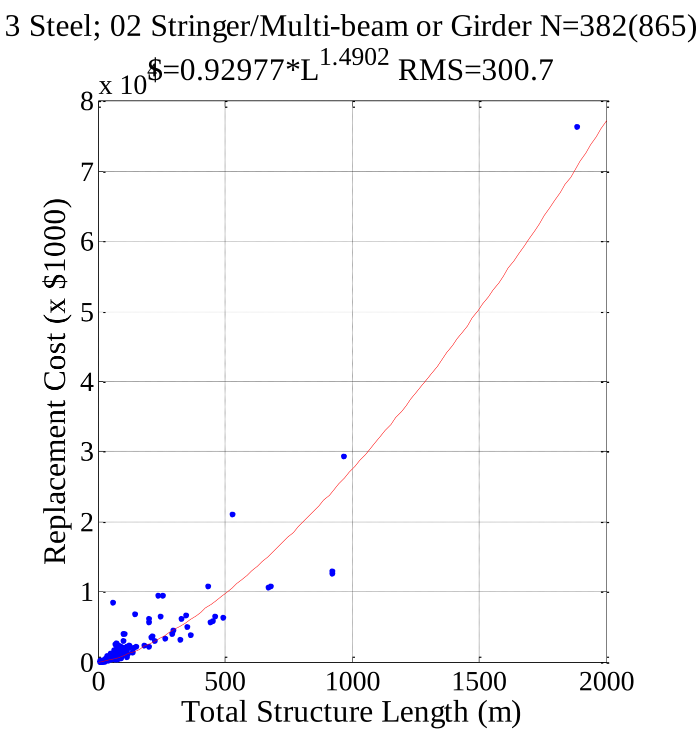


Figure C.9: Cost Model 16.



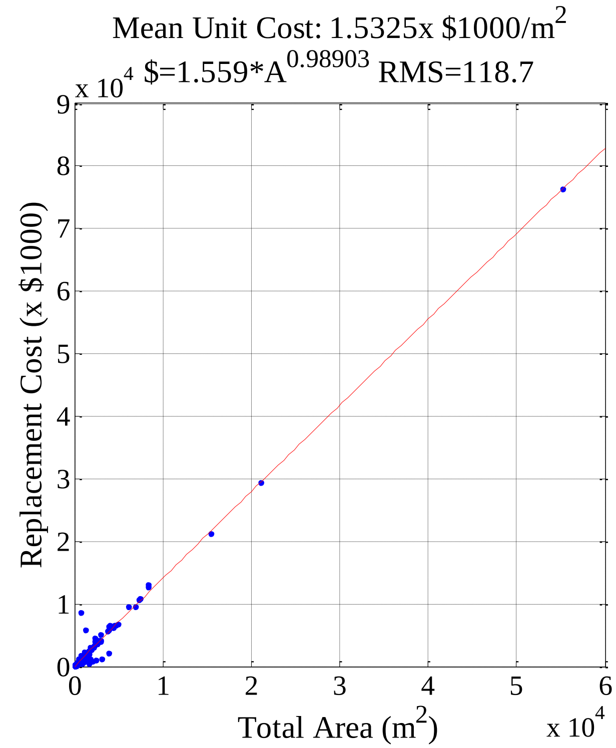


Figure C.10: Cost Model 18.

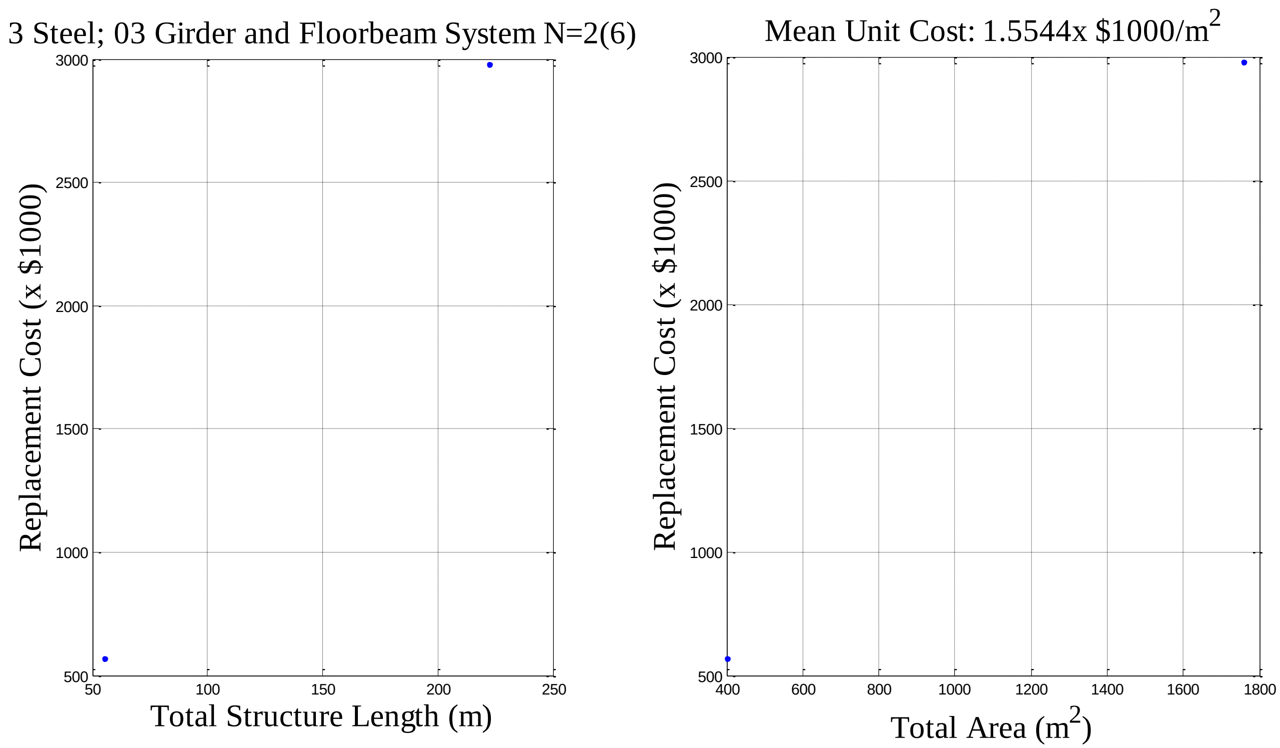


Figure C.11: Cost Model 19.

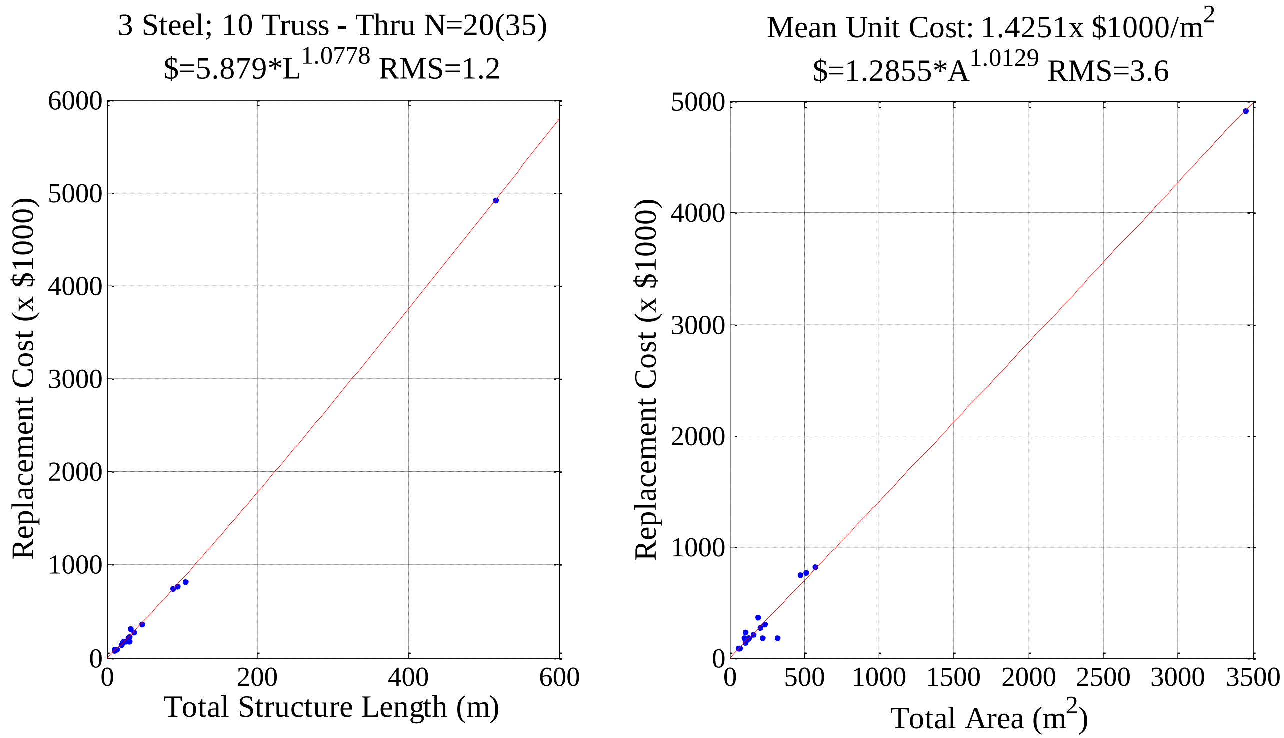


Figure C.12: Cost Model 21.

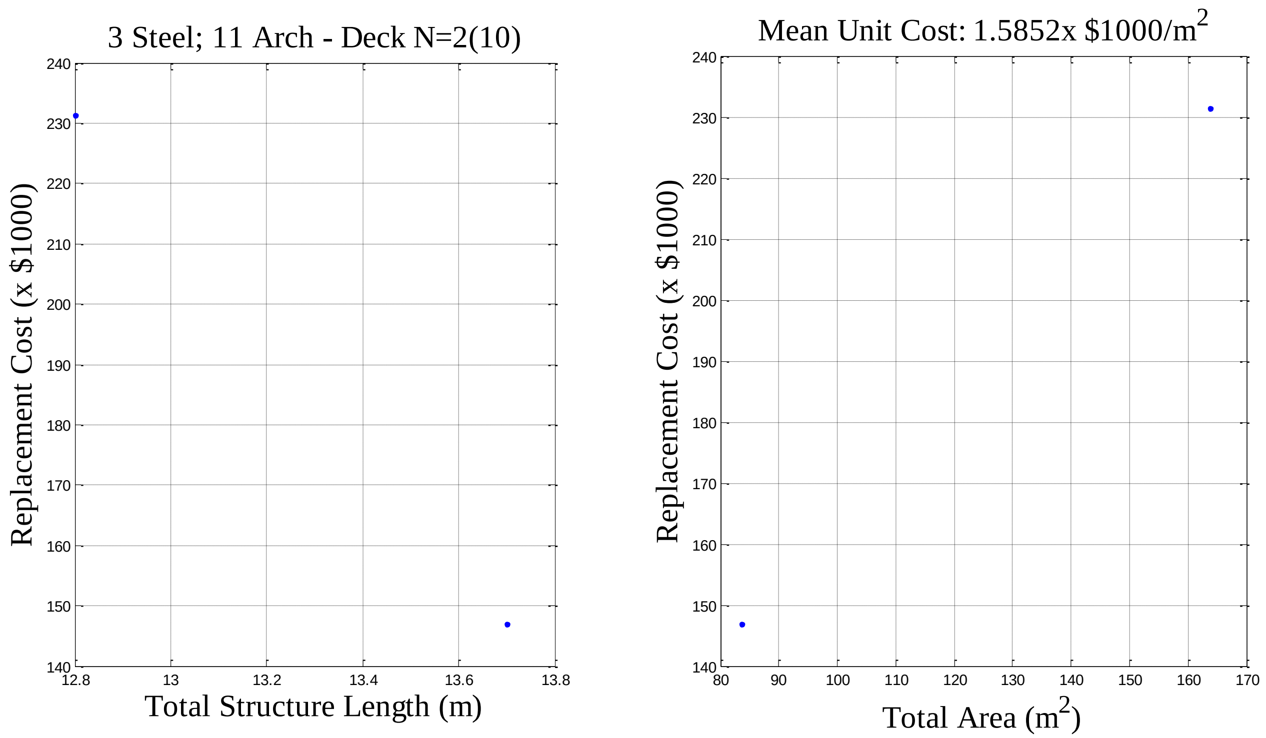


Figure C.13: Cost Model 22.

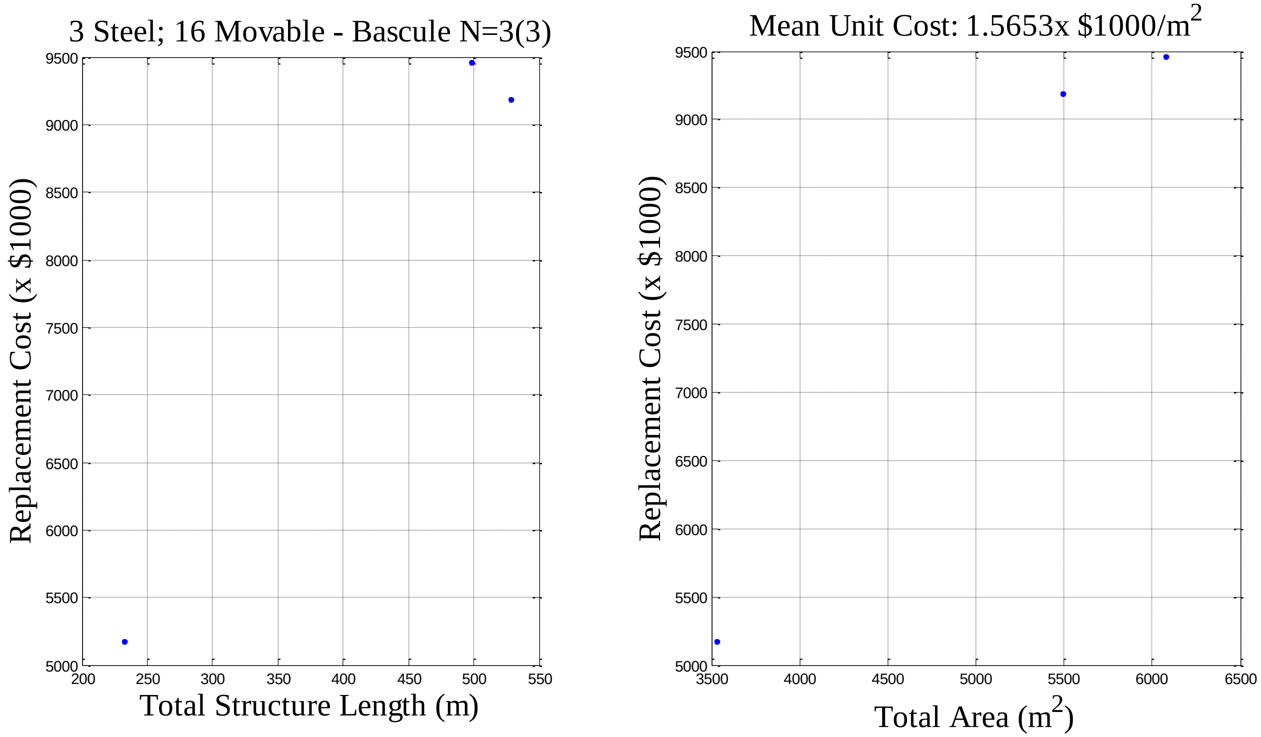


Figure C.14: Cost Model 23.

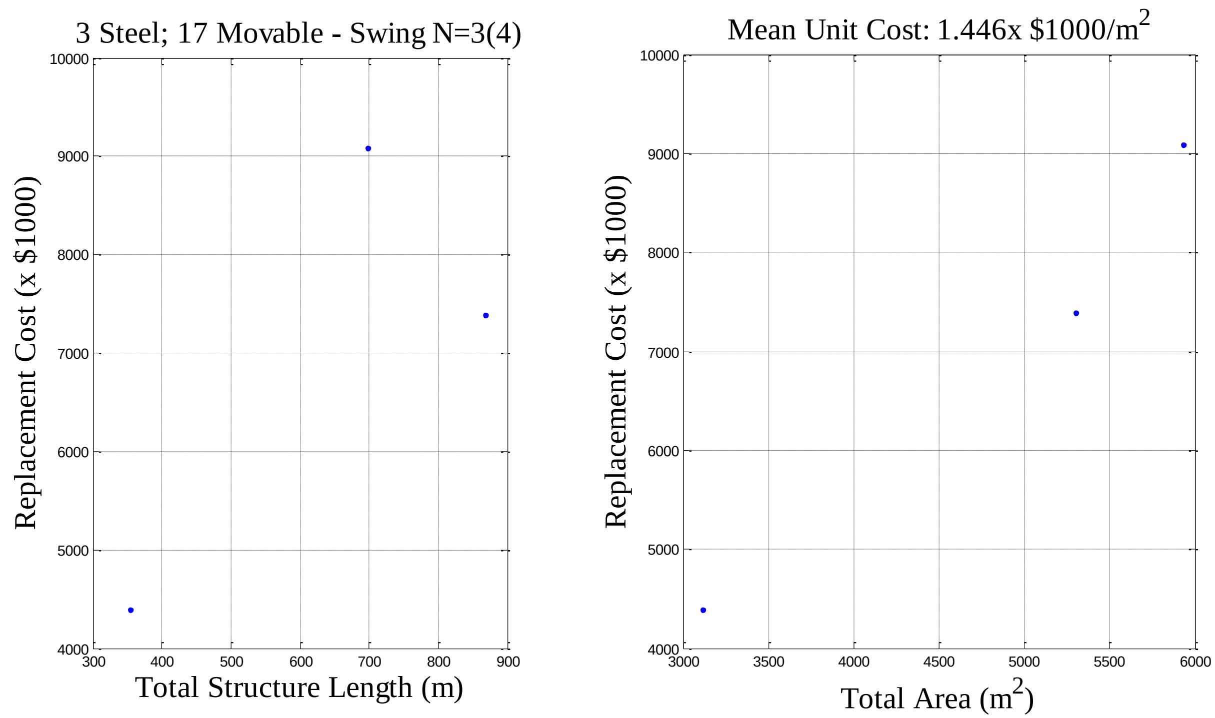


Figure C.15: Cost Model 24.

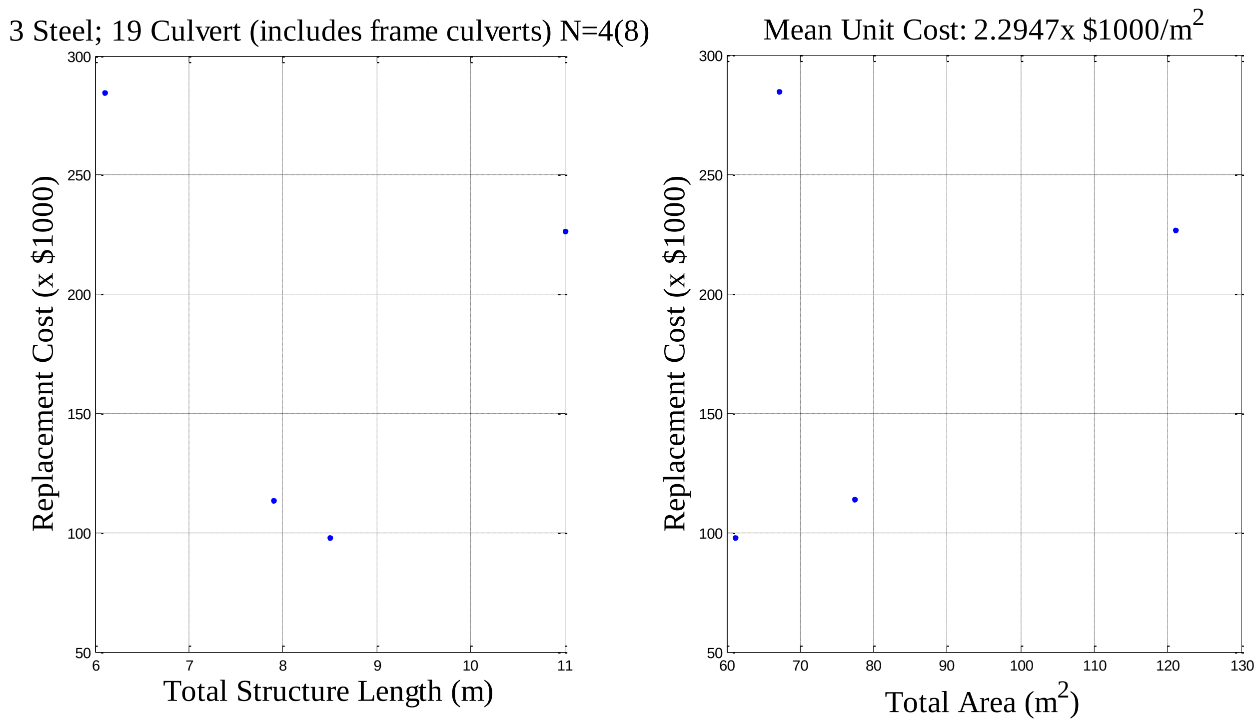


Figure C.16: Cost Model 25.

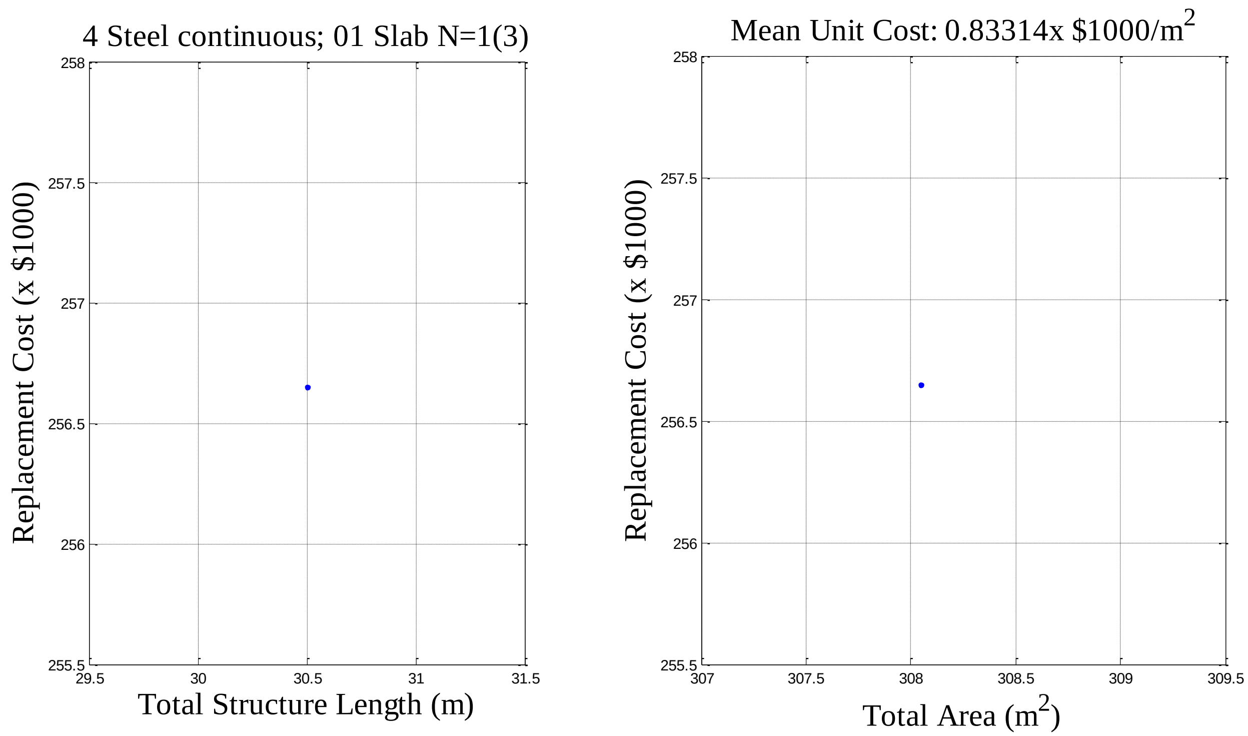


Figure C.17: Cost Model 27.

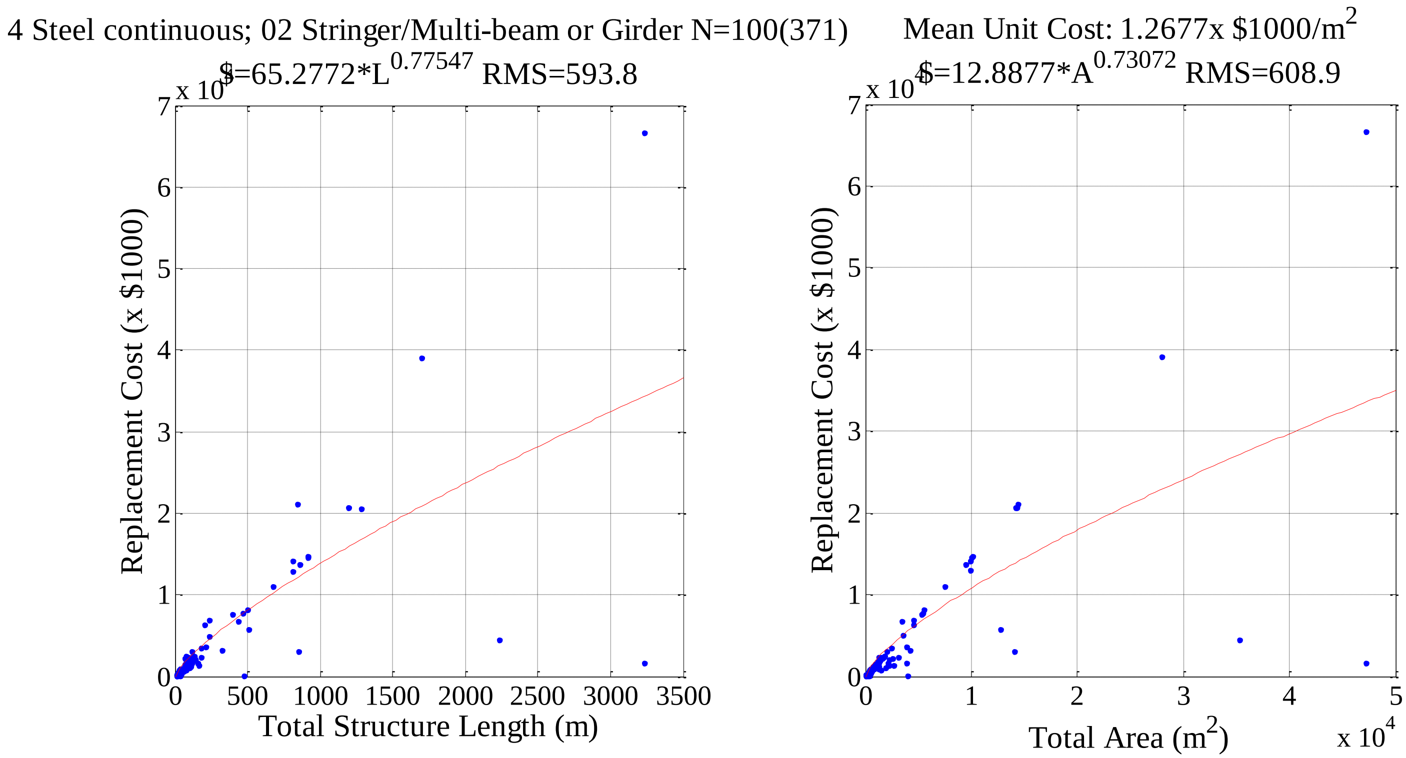


Figure C.18: Cost Model 28.

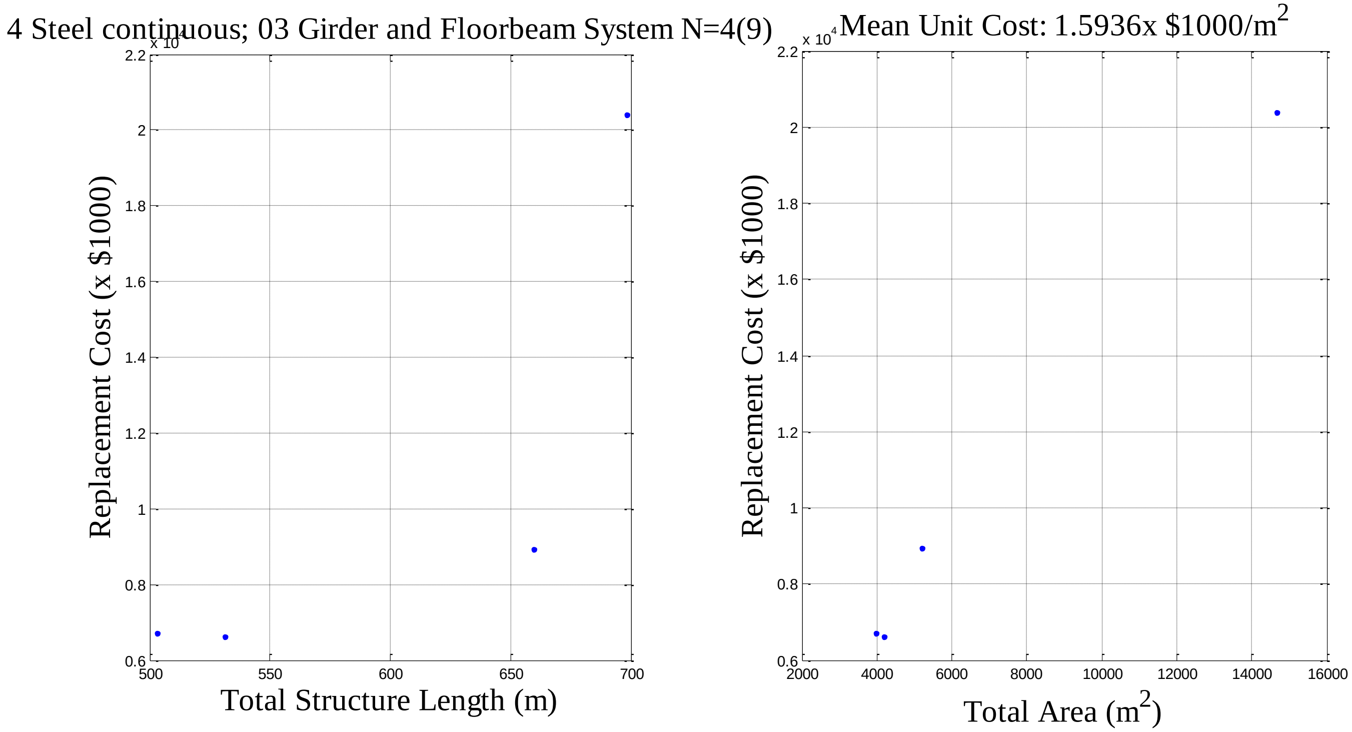


Figure C.19: Cost Model 29.

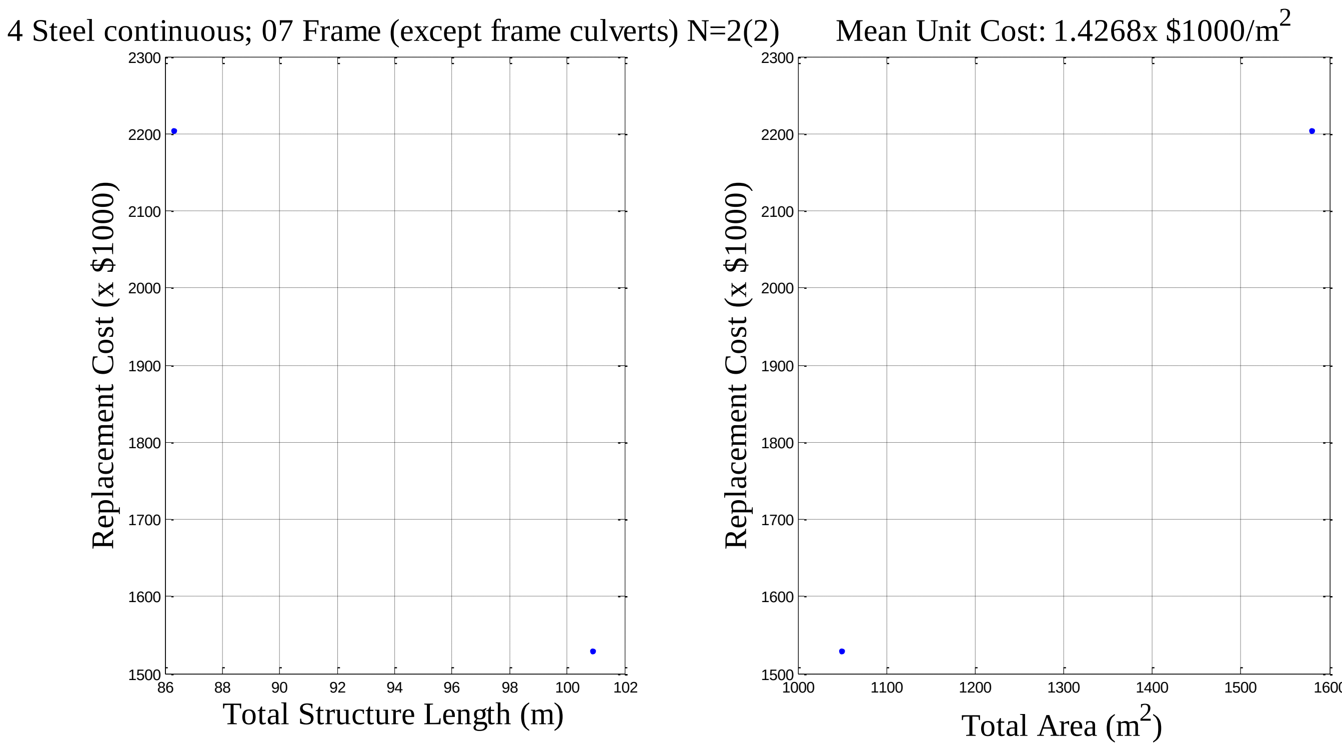


Figure C.20: Cost Model 30.

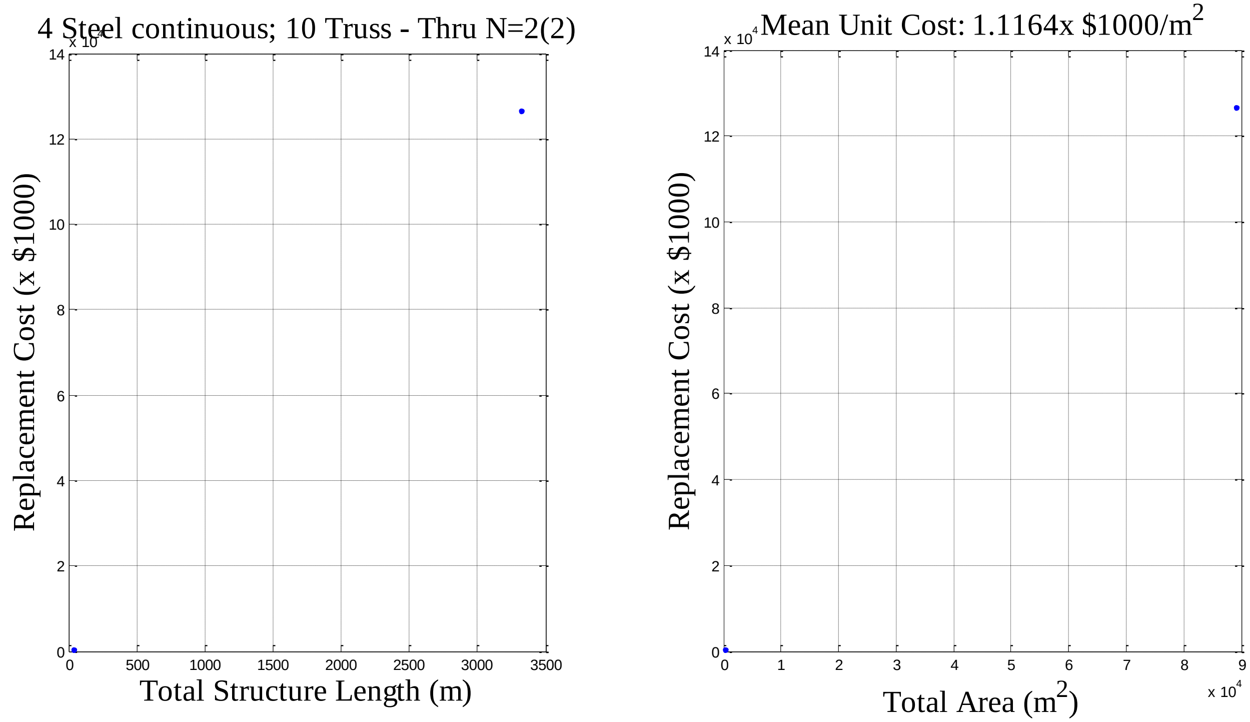


Figure C.21: Cost Model 31.

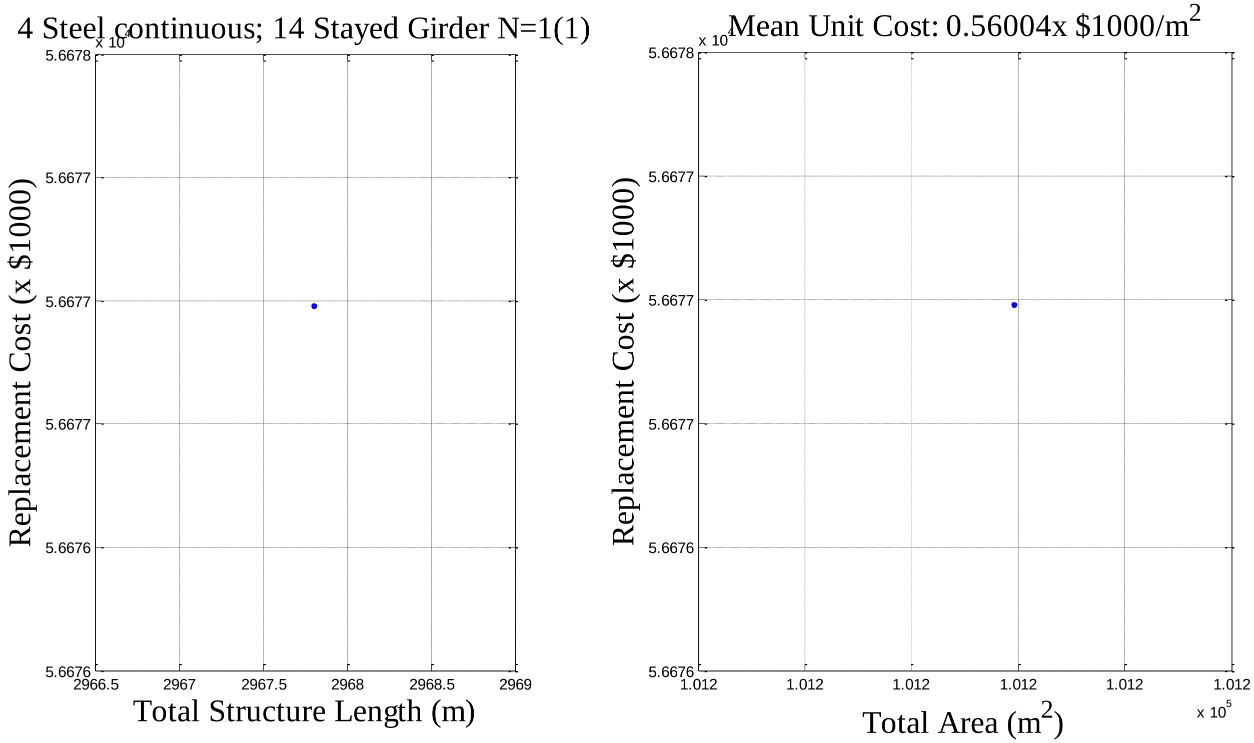


Figure C.22: Cost Model 32.

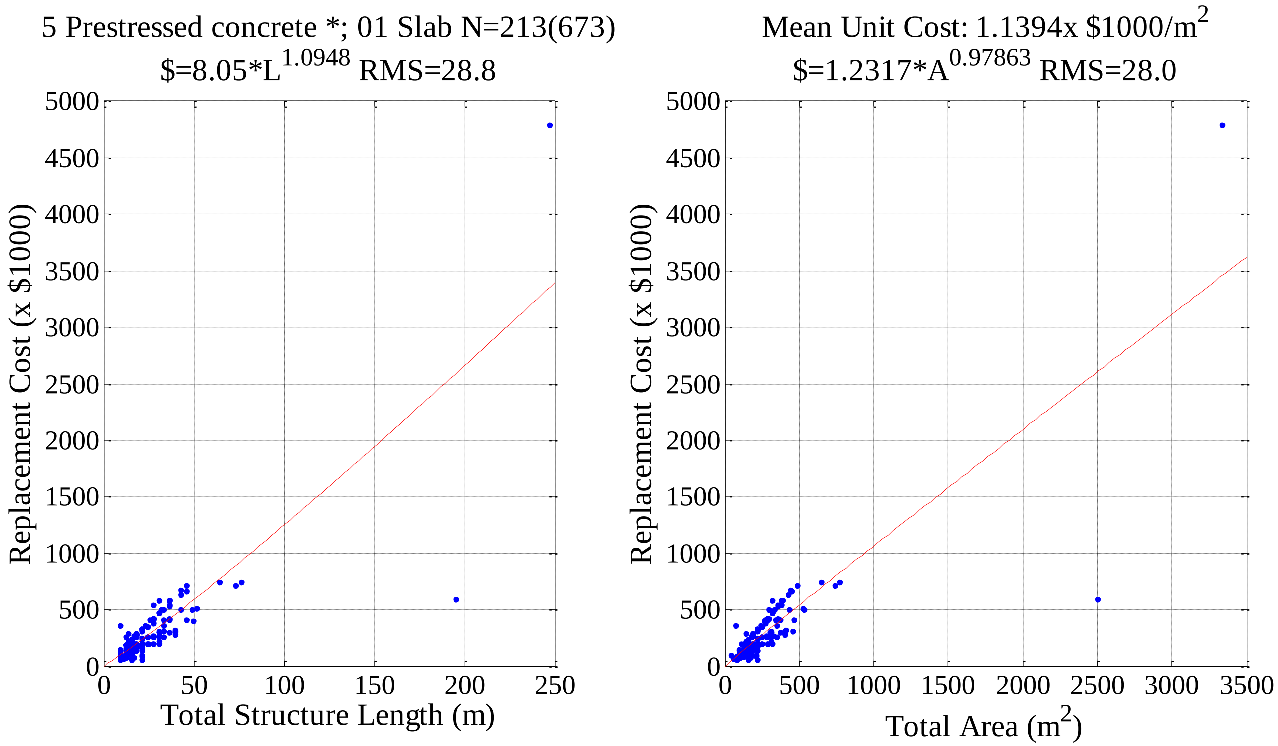


Figure C.23: Cost Model 34.

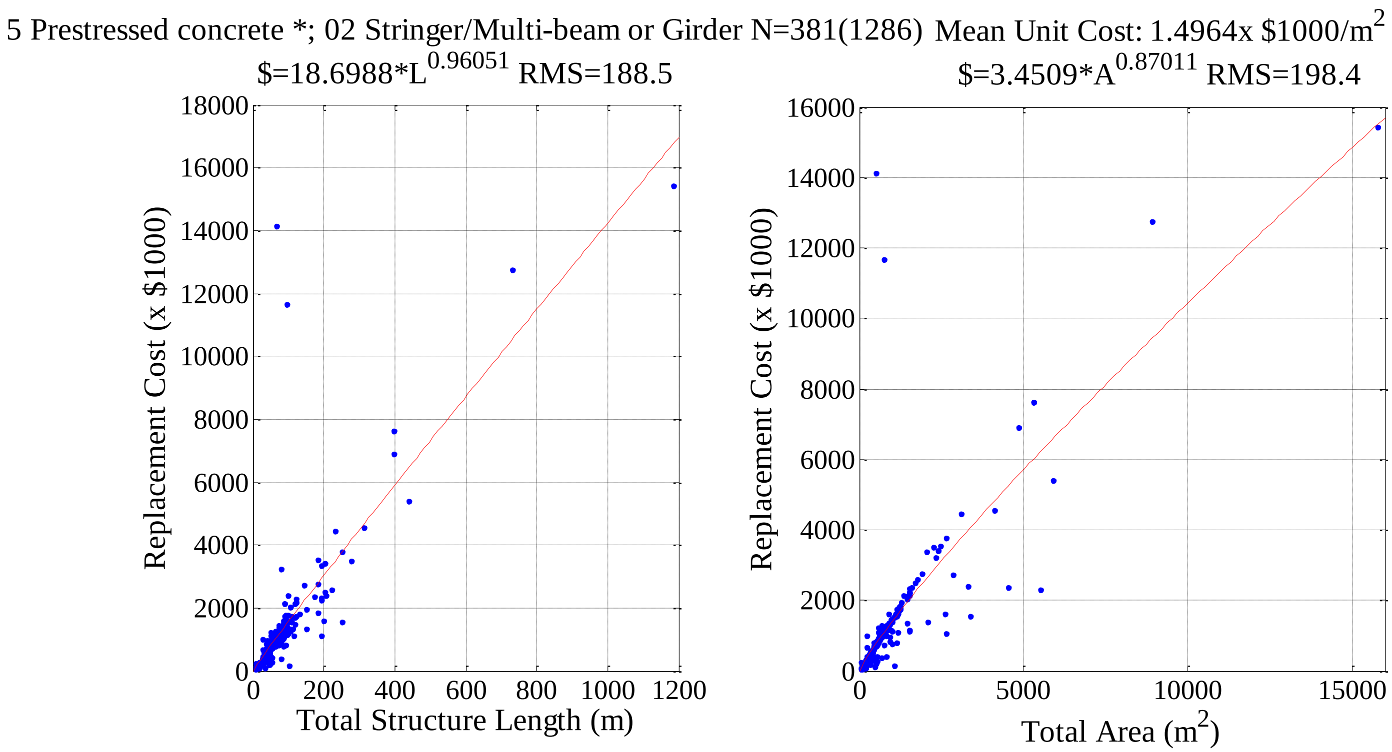


Figure C.24: Cost Model 35.

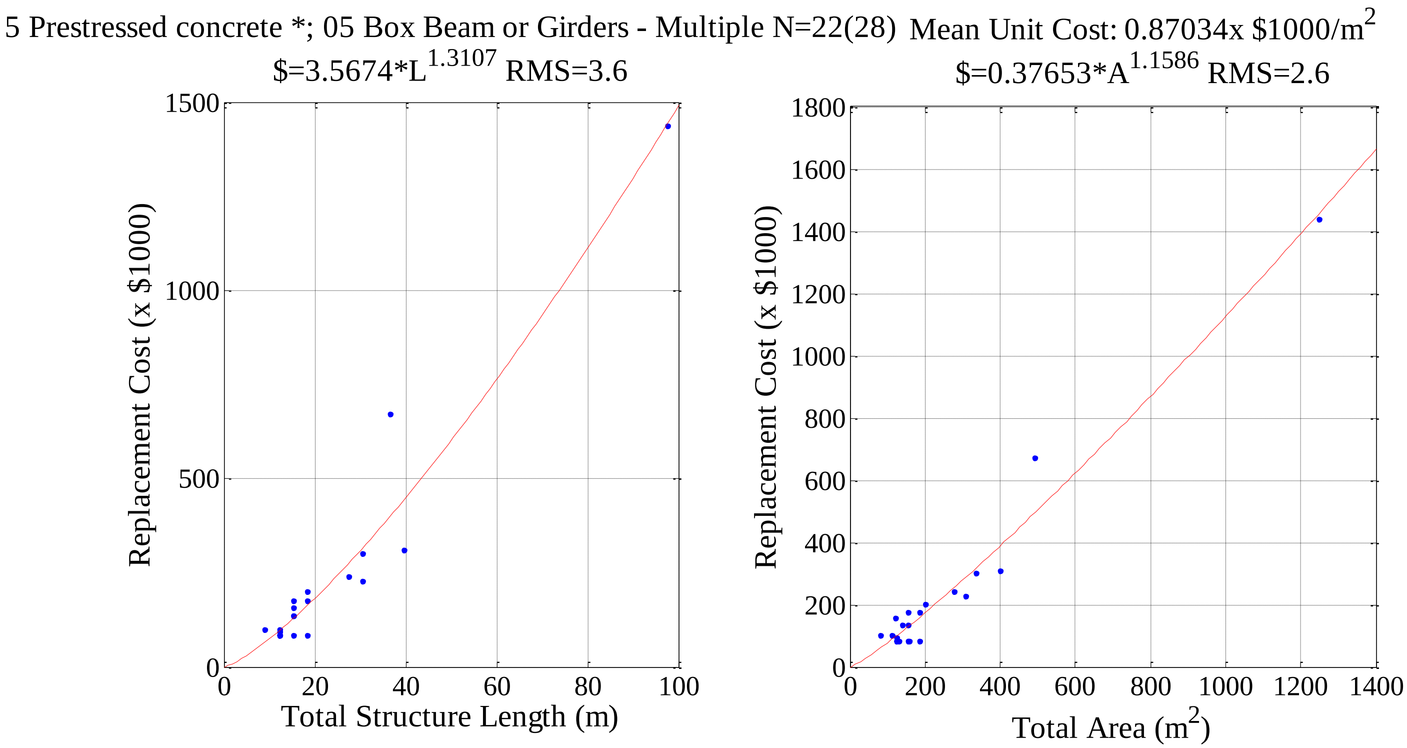


Figure C.25: Cost Model 38.

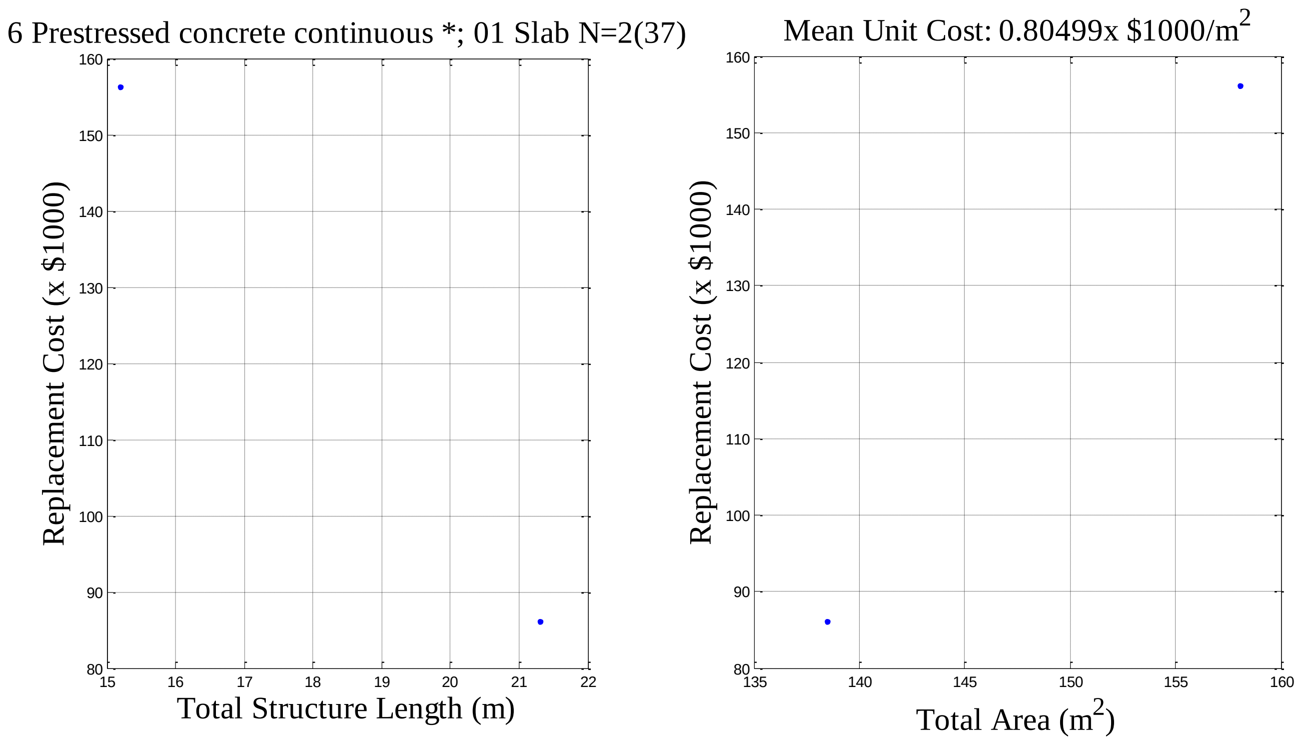


Figure C.26: Cost Model 41.

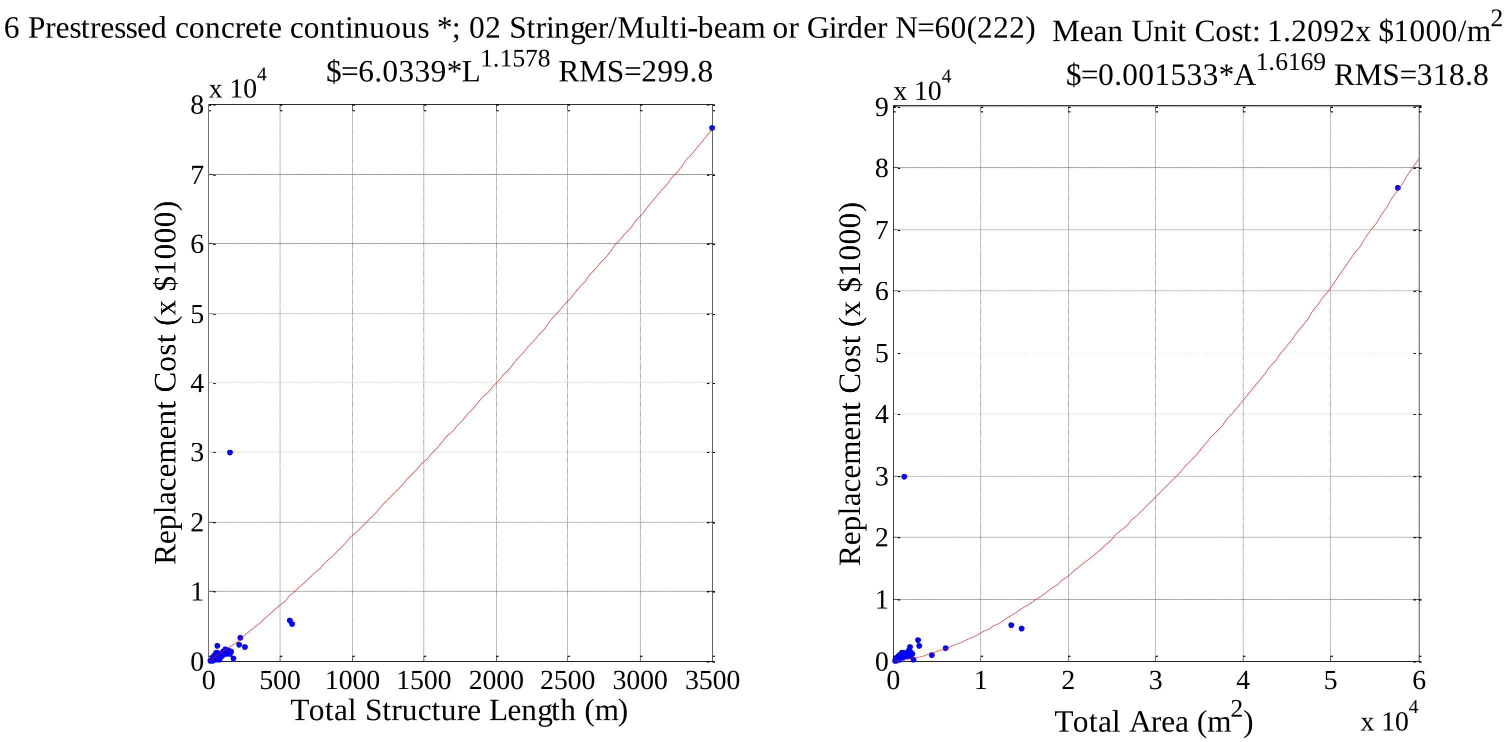


Figure C.27: Cost Model 42.

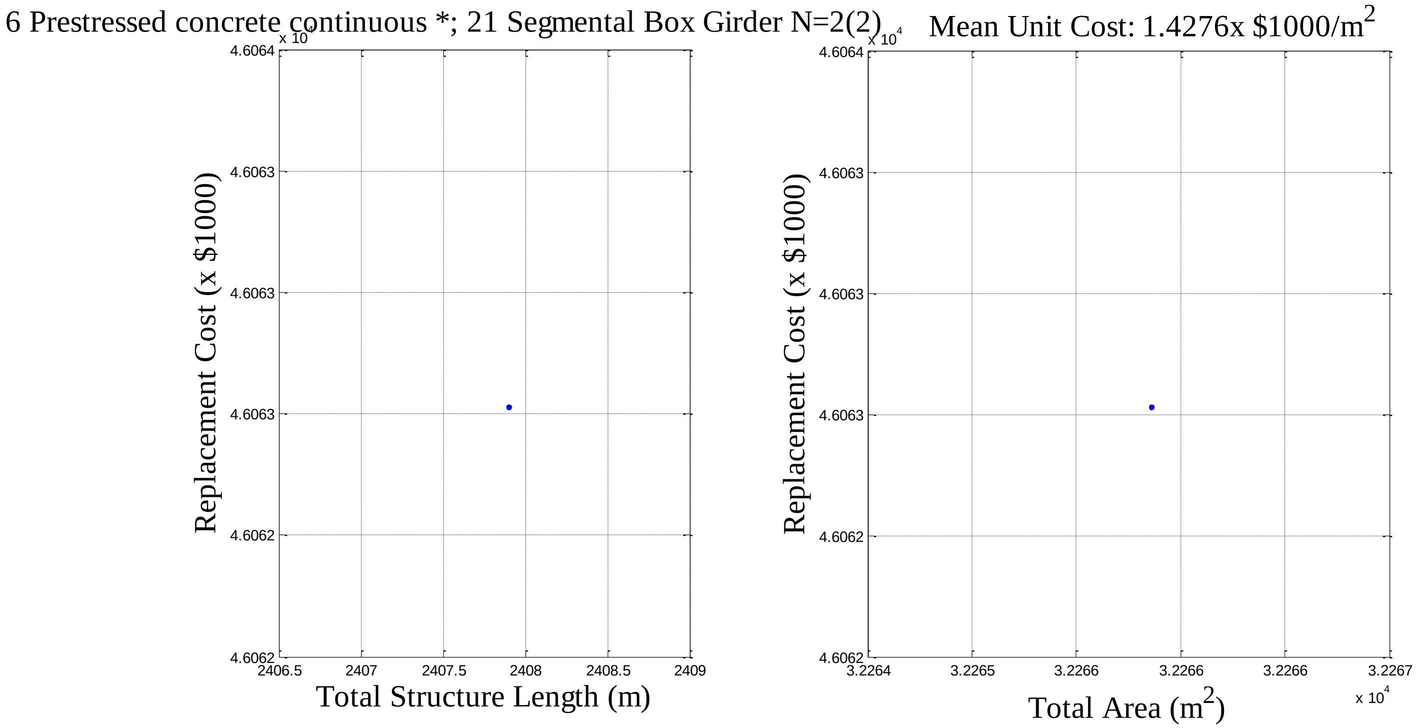


Figure C.28: Cost Model 43.

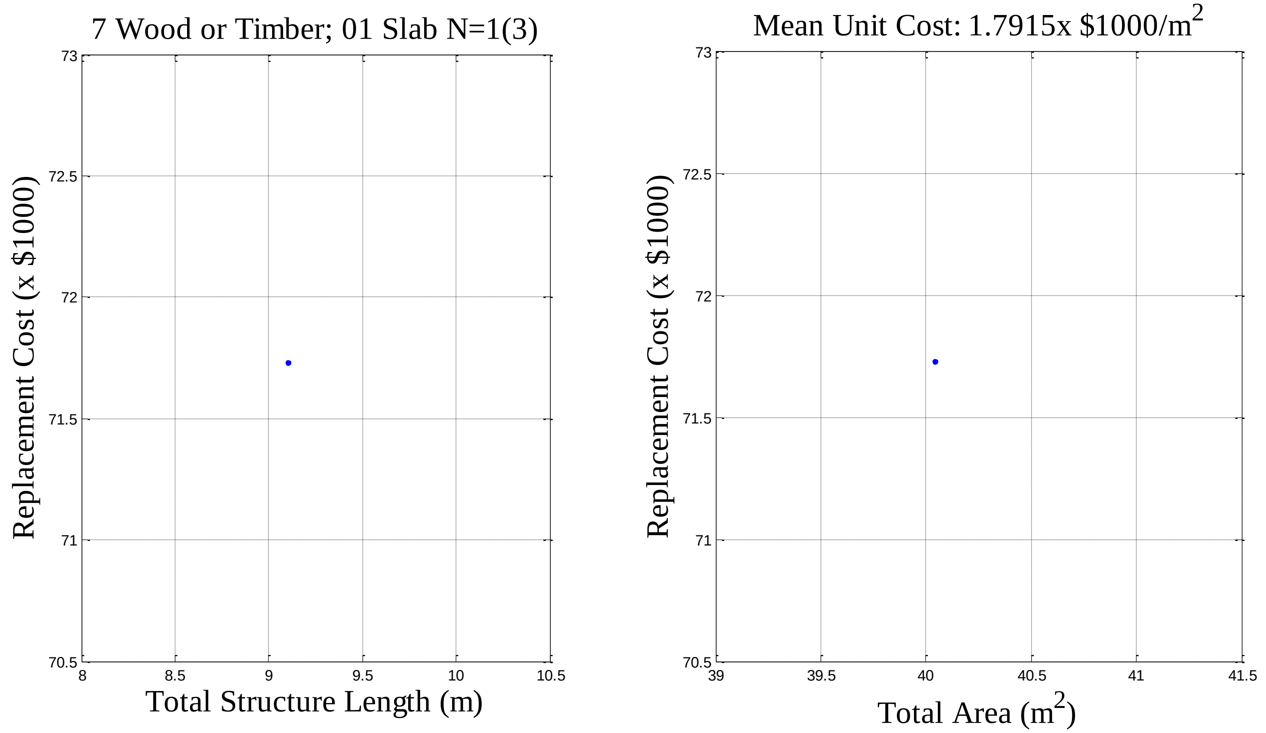


Figure C.29: Cost Model 44.

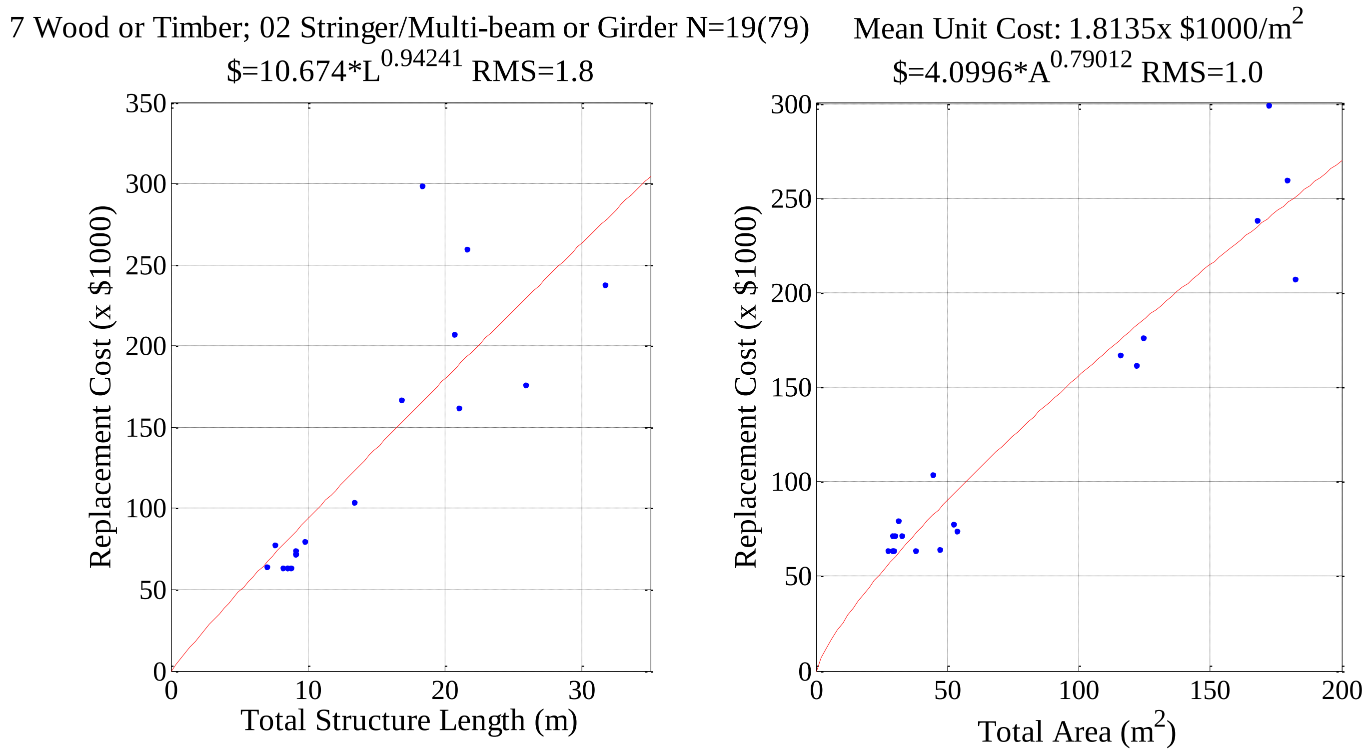


Figure C.30: Cost Model 45.

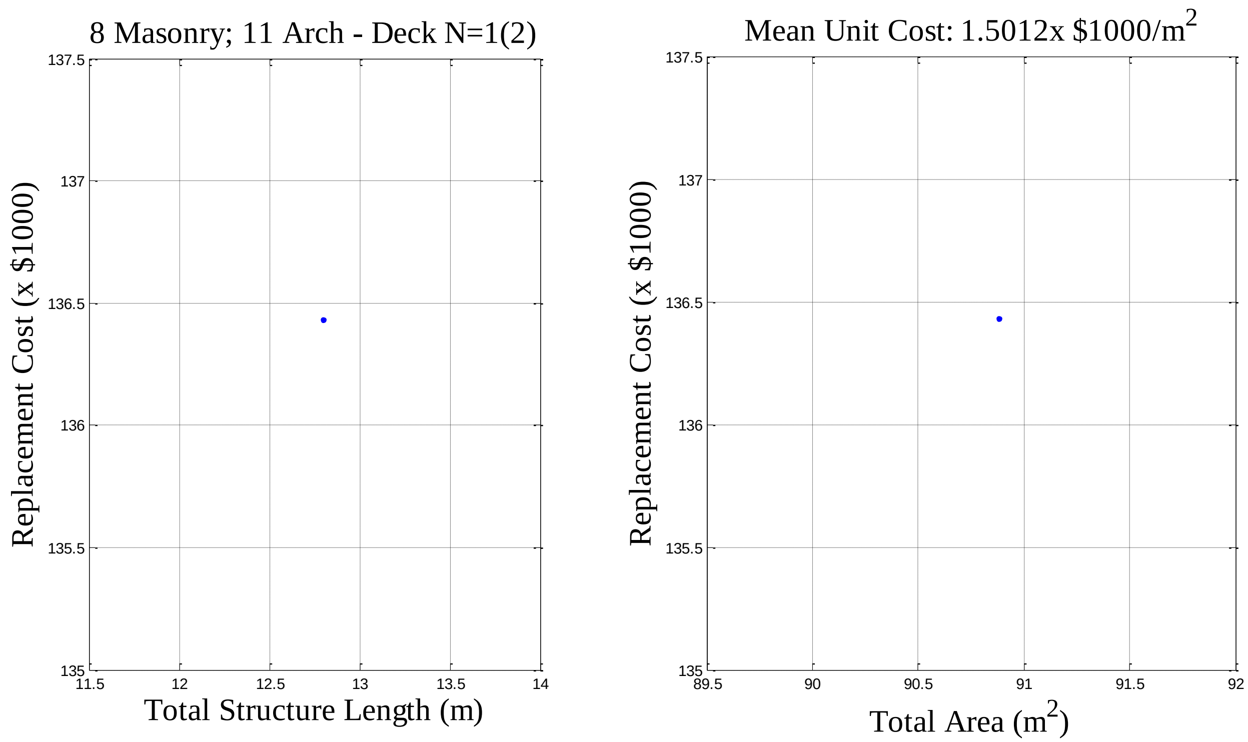


Figure C.31: Cost Model 46.

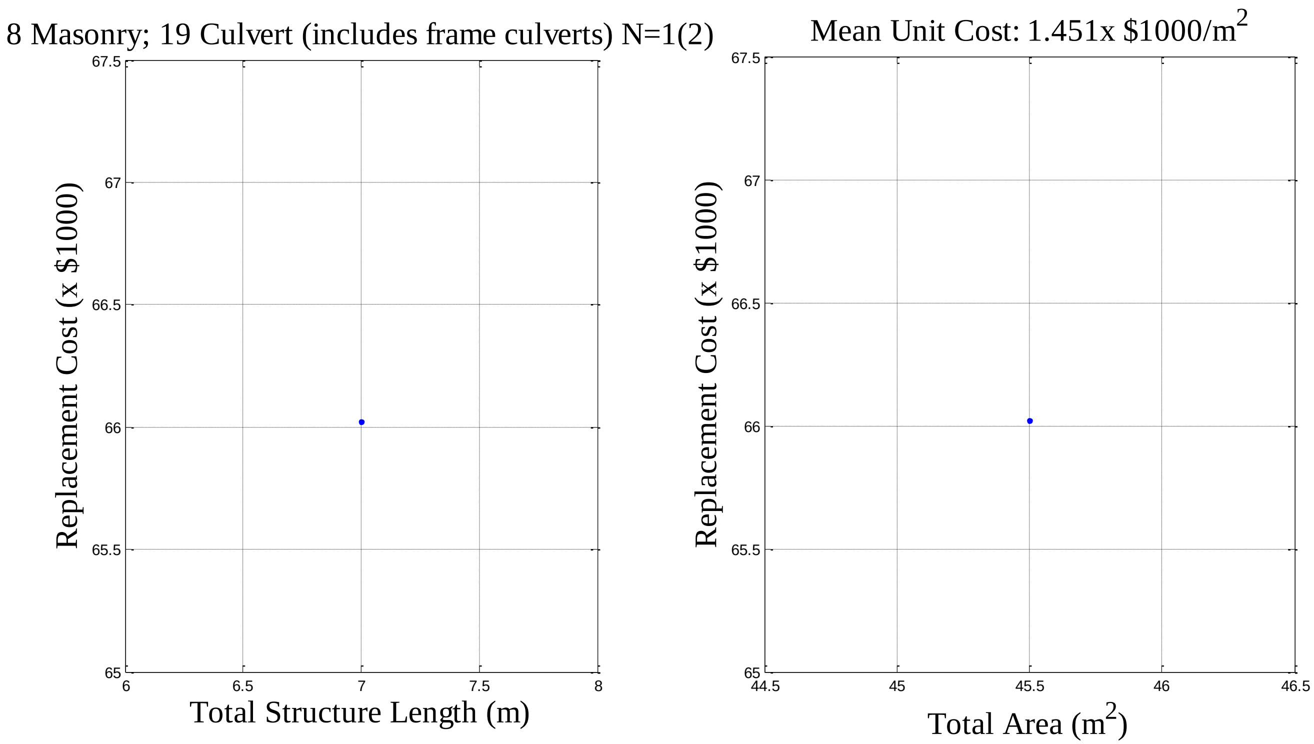


Figure C.32: Cost Model 47.

9 Aluminum, Wrought Iron, or Cast Iron; 19 Culvert (includes frame culverts) N=4(10) Mean Unit Cost: 2.5728x $1000/m^2

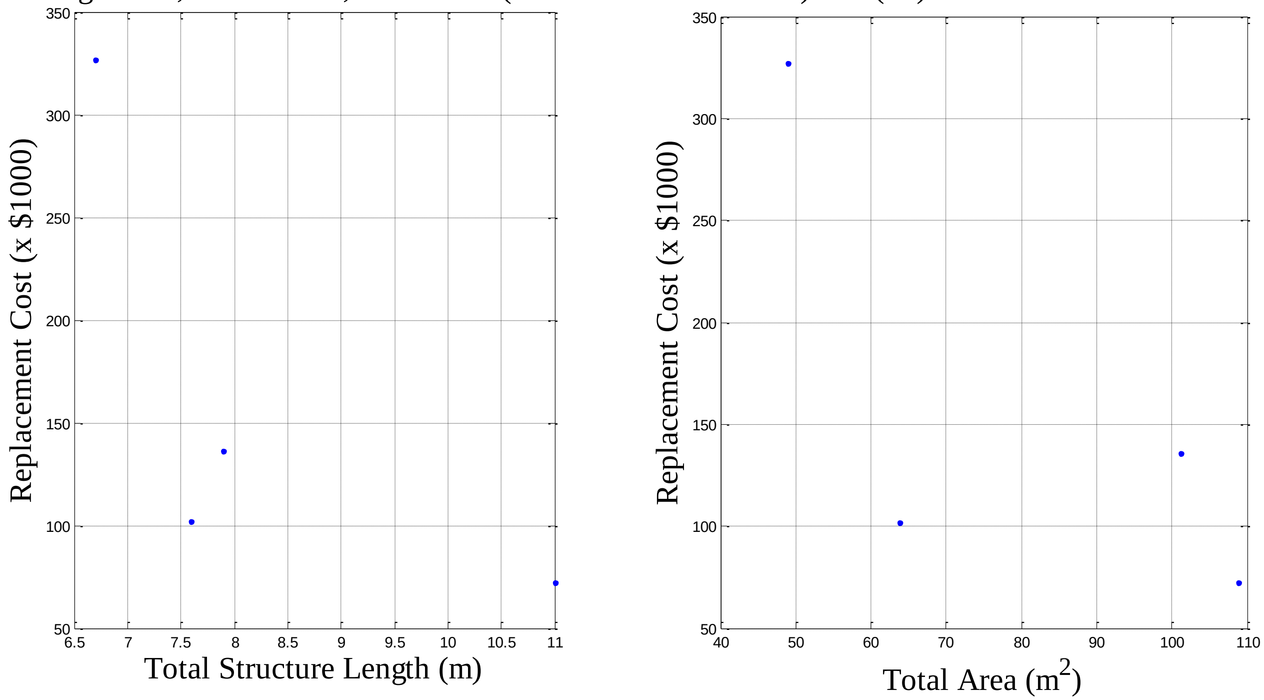


Figure C.33: Cost Model 48.

<u>Bridge Replacement Cost Models Assignments</u>

For bridge groups that were unable to establish a cost model or unit area cost, a cost model or unit area cost from a similar bridge group was assigned to this group.

Table C.2: Bridge Cost Models Assignment.

Bridge Groups Without A Cost Model or Unit Area Cost			Assigned Cost Model[b]
Cost Model Number	Material Type	Structure Type	Cost Model Number
3	Concrete	Girder and Floor Beam System	4
5	Concrete	Box Beam or Girders - Multiple	
6	Concrete	Frame (except Frame Culverts)	
8	Concrete	Tunnel	
10	Concrete	Channel Beam	
11	Concrete	Other	
15	Concrete Continuous	Box Beam or Girders - Multiple	16
17	Steel	Slab	18
20	Steel	Frame (except Frame Culverts)	
26	Steel	Other	
33	Steel Continuous	Movable - Swing	31
36	Prestressed Concrete	Girder and Floor Beam System	35
37	Prestressed Concrete	Tee Beam	
39	Prestressed Concrete	Channel Beam	
40	Prestressed Concrete	Other	

(b) For cost model details refer to Table 7.1.

Appendix D

SCDOT Maintenance Cost Schedule from Jul 2010 to June 2011

SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION
HIGHWAY MAINTENANCE MANAGEMENT SYSTEM
Maintenance Work Description Cost Distribution By StateWide

Interstate → every month
Primary → " six months
Secondary → " year

Organization Unit:99010 - STATEWIDE MAINTENANCE SUMMARY Fiscal Year:JULY10-JUNE11 From Month:JUL 2010 To Month: JUN 2011

Activity	Accomplishment		Cost Distribution				
	Amount	UOM	Labor	Equipment	Material	Total	Unit Cost
102 - SURFACE REPAIRS							
MAJOR LEVELING/STRENGTHENING	47,683.626	TONS	1,338,208	658,895	2,984,960	4,982,063	104.48
PATCHING/MINOR LEVELING	49,533.611	TONS	7,894,443	2,464,493	4,099,551	14,458,487	291.89
RESURFACE(REQUEST ONLY)	3,504.500	TONS	87,366	37,291	225,569	350,226	99.94
107 - CHIP SEAL							
DOUBLE Contract?	98,426.000	SQ YDS	38,682	21,793	56,812	117,287	1.19
SINGLE	5,670,949.548	SQ YDS	616,084	279,017	5,219,921	6,115,022	1.08
TRIPLE	637.000	SQ YDS	67,854	37,406	182,828	288,088	452.26
108 - MILLING							
	26,308.830	SQ YDS	72,736	45,429	11,923	130,088	4.94
110 - BASE REPAIR							
FULL DEPTH ASPHALT	252,395.904	SQ YDS	2,559,288	1,242,293	3,197,305	6,998,886	27.73
FULL DEPTH CONCRETE	1,443.830	SQ YDS	39,753	15,474	17,550	72,777	50.41
RECLAMATION	197,254.840	SQ YDS	412,512	256,797	630,627	1,299,936	6.59
SPALL REPAIR	306.610	SQ YDS	5,702	2,252	2,648	10,602	34.58
120 - CRACK SEAL PAVEMENT							
ASPHALT	15.840	LN MI	25,321	5,435	15,479	46,235	2,918.88
CONCRETE	0.000	LN MI					.00
130 - MACHINE EARTH ROADS							
	2,260.502	MILES	213,748	121,057	94,340	429,145	189.84
DUST CONTROL	0.700	MILES	423	180	133	736	1,051.43
MACHINE EARTH ROADS	1,364.100	MILES	152,949	82,843	91,508	327,300	239.94
202 - SLOPES							
INSTALL/MAINTAIN	7,659.590	SQ YDS	28,949	11,205	18,788	58,942	7.70

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Figure D.1 SCDOT Maintenance Cost Schedule 1

SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION
HIGHWAY MAINTENANCE MANAGEMENT SYSTEM

Maintenance Work Description Cost Distribution By StateWide

Organization Unit:99010 - **STATEWIDE MAINTENANCE SUMMARY** Fiscal Year:**JULY10-JUNE11** From Month:**JUL 2010** To Month: **JUN 2011**

Activity	Accomplishment		Cost Distribution				
	Amount	UOM	Labor	Equipment	Material	Total	Unit Cost
REPAIR	52,389.080	SQ YDS	231,463	105,189	46,295	382,947	7.31
203 - SHOULDERS/DITCHES							
CLEAN OUTFALL	372,679.182	LF	558,333	268,235	15,147	841,715	2.26
CONSTRUCT OUTFALL	26,410.400	LF	35,431	12,464	7,256	55,151	2.09
REGRADE ROADSIDE DITCH	7,608,997.890	LF	3,987,270	1,727,357	16,894	5,731,521	.75
REGRADE SHOULDER & DITCH	15,139,278.738	LF	3,045,244	1,563,597	28,958	4,637,799	.31
REGRADE/REPAIR SHOULDER	17,650,061.512	LF	2,441,303	1,166,394	140,730	3,748,427	.21
REPAIR SHOULDER	1,620,641.215	LF	872,478	334,266	300,332	1,507,076	.93
WIDEN SHOULDER	121,004.600	LF	128,300	63,683	8,951	200,934	1.66
204 - ROAD WIDEN/SHOULDER PAVIN							
BIKELANES	3.730	SH MI	15,292	8,982	66,976	91,250	24,463.81
CROSSOVER	0.080	SH MI	4,291	2,079	1,847	8,217	102,712.50
SHOULDER PAVING	2,060.950	SH MI	60,466	27,907	136,533	224,906	109.13
TURN LANES	0.390	SH MI	7,593	4,695	17,408	29,696	76,143.59
305 - DRAINAGE STRUCTURES							
CLEAN	26,589.000	EACH	1,141,242	378,937	3,717	1,523,896	57.31
INSTALL	274.000	EACH	131,210	45,193	22,454	198,857	725.76
REPAIR	2,114.500	EACH	779,274	265,292	195,000	1,239,566	586.22
UPGRADE	260.000	EACH	122,862	43,560	22,798	189,220	727.77
306 - DRAINAGE PIPE							
CLEAN	348,479.600	LF	943,584	405,621	4,602	1,353,807	3.88
INSTALL	8,289.800	LF	256,686	108,751	113,131	478,568	57.73
REMOVE	1,230.000	LF	23,879	11,884	5,040	40,803	33.17

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Figure D.2 SCDOT Maintenance Cost Schedule 2

SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION
HIGHWAY MAINTENANCE MANAGEMENT SYSTEM

Maintenance Work Description Cost Distribution By StateWide

Organization Unit:99010 - STATEWIDE MAINTENANCE SUMMARY Fiscal Year:JULY10-JUNE11 From Month:JUL 2010 To Month: JUN 2011

Activity	Accomplishment		Cost Distribution				
	Amount	UOM	Labor	Equipment	Material	Total	Unit Cost
REPAIR	19,694.730	LF	637,598	263,622	163,102	1,064,322	54.04
401 - MOWING 59% of mowing is contract							
BRUSH MANAGEMENT 10% is interstate	20,481.444	ACRES	1,452,330	715,380		2,167,710	105.84
HAND TRIM	877.921	ACRES	664,851	158,881		823,732	938.28
OUTDOOR ADVERTISING WINDOW	0.560	ACRES	1,008	144		1,152	2,057.14
ROUTINE 194530 total acres	83,647.654	ACRES	1,219,136	726,639		1,945,775	23.26
SAFETY — sight distance	12,213.741	ACRES	353,076	171,979		525,055	42.99
402 - HERBICIDE APPLICATION							
100 - TOTAL VEGETATION CONTROL	2,909.797	ACRES	193,064	57,437	43,242	293,743	100.95
200 - BRUSH	2,308.034	ACRES	51,045	20,751	173,693	245,489	106.36
300 - TREES	1,749.828	ACRES	70,299	17,527	103,296	191,122	109.22
400 - BROADLEAF WEEDS	3,404.941	ACRES	73,349	30,720	68,696	172,765	50.74
500 - GRASSY WEEDS	1,313.411	ACRES	24,175	9,654	16,487	50,316	38.31
600 - TURF	5,287.730	ACRES	38,228	17,370	29,135	84,733	16.02
403 - GRASSING							
	208.994	ACRES	53,388	17,333	38,988	109,709	524.94
405 - LIMB MANAGEMENT							
	22,637.083	SH MI	3,586,766	1,348,513	16	4,935,295	218.02
406 - HIGHWAY BEAUTIFICATION							
LANDSCAPING	153.887	ACRES	61,609	13,796		75,405	490.00
WILDFLOWER	20.500	ACRES	1,133	170		1,303	63.56
407 - LITTER CONTROL							
DEAD ANIMALS	1,105,978.085	LBS	762,379	211,044	1,694	975,117	.88

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Figure D.3 SCDOT Maintenance Cost Schedule 3

SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION
HIGHWAY MAINTENANCE MANAGEMENT SYSTEM

Maintenance Work Description Cost Distribution By StateWide

Organization Unit:99010 - **STATEWIDE MAINTENANCE SUMMARY** Fiscal Year:**JULY10-JUNE11** From Month:**JUL 2010** To Month: **JUN 2011**

Activity	Accomplishment		Cost Distribution				
	Amount	UOM	Labor	Equipment	Material	Total	Unit Cost
LITTER	3,489,180.440	LBS	1,981,591	496,751	4,678	2,483,020	.71
LITTER BAG PICKUP	788,675.840	LBS	243,112	56,042	1,721	300,875	.38
408 - TREE REMOVAL							
FALLEN	21,803.500	EACH	1,954,374	599,352		2,553,726	117.12
STANDING	19,132.000	EACH	1,149,201	377,343		1,526,544	79.79
409 - DEBRIS REMOVAL							
CONSTRUCTION/DEMOLITION	3,688.300	CU YDS	38,448	15,831		54,279	14.72
HAZARDOUS WASTE	154.900	CU YDS	2,534	667		3,201	20.66
PERSONAL PROPERTY/HOUSEHLD ITM	397.790	CU YDS	14,446	3,608		18,054	45.39
SOIL, MUD, SAND	7,970.061	CU YDS	88,869	34,499		123,368	15.48
VEGETATION	12,228.931	CU YDS	288,722	95,086		383,808	31.39
VEHICLES/VESSELS	364.320	CU YDS	7,026	2,005		9,031	24.79
WHITE GOODS	4.150	CU YDS	356	50		406	97.83
410 - ROADWAY CLEANING							
CLEAN BY HAND	1,673,467.070	LF	355,448	85,711		441,159	.26
CLEAN BY MACHINE	10,402,073.418	LF	438,213	216,877		655,090	.06
460 - SECRETARY OF TRANSPORTATI							
ENHANCEPROJ-STREETSCAPING	0.000	EACH	16			16	.00
501 - DRIVEWAYS							
INSTALL	1,294.000	EACH	707,624	292,518	377,154	1,377,296	1,064.37
MAINTENANCE	7,356.340	EACH	2,120,431	854,507	1,091,164	4,066,102	552.73
REMOVE	595.000	EACH	40,822	16,382	1,116	58,320	98.02

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Figure D.4 SCDOT Maintenance Cost Schedule 4

SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION
HIGHWAY MAINTENANCE MANAGEMENT SYSTEM
Maintenance Work Description Cost Distribution By StateWide

Organization Unit:99010 - STATEWIDE MAINTENANCE SUMMARY Fiscal Year:JULY10-JUNE11 From Month:JUL 2010 To Month: JUN 2011

Activity	Accomplishment		Cost Distribution				
	Amount	UOM	Labor	Equipment	Material	Total	Unit Cost
504 - CONCRETE STRUCTURES							
CURB RAMP - INSTALL	837.930	LF	19,307	4,383	3,202	26,892	32.09
INSTALL	2,179.000	LF	87,375	23,873	24,675	135,923	62.38
REMOVE	3,291.000	LF	61,182	25,264	1,653	88,099	26.77
REPAIR	38,025.600	LF	83,475	23,756	10,598	117,829	3.10
SIDEWALK-REPAIR	216,906.010	LF	292,238	92,025	51,491	435,754	2.01
603 - SIGNS							
DELIVER	43,386.000	EACH	34,187	12,026	1,098	47,311	1.09
MAINTAIN/REPLACE	232,417.000	EACH	4,459,749	953,843	2,276,190	7,689,782	33.09
MANUFACTURE	68,707.000	EACH	499,201	14,134	4,344	517,679	7.53
NEW INSTALL	6,714.000	EACH	185,825	42,789	312,582	541,196	80.61
REVISE (BY REQUEST ONLY)	378.000	EACH	9,964	1,878	5,825	17,667	46.74
TEMPORARY	2,323.000	EACH	58,201	10,873	26,252	95,326	41.04
604 - TRAFFIC SIGNAL							
CONTRACT INSPECTION	1,167.000	EACH	79,487	12,095	186	91,768	78.64
NEW INSTALL	49.000	EACH	42,412	9,196	10,349	61,957	1,264.43
PREVENTATIVE MAINT INSPECTION	2,728.700	EACH	155,865	41,609	7,295	204,769	75.04
REBUILD	56.000	EACH	60,518	9,970	29,300	99,788	1,781.93
REPAIR	5,784.500	EACH	716,676	150,426	154,345	1,021,447	176.58
REVISE	801.000	EACH	181,867	38,418	73,566	293,851	366.86
TROUBLE CALL(AFTER HOURS)	1,620.000	EACH	123,605	32,502	36,628	192,735	118.97
605 - FLASHERS							
CONTRACT INSPECTION	6.000	EACH	408	63		471	78.50
INSTALL	57.000	EACH	31,628	6,969	67,875	106,472	1,867.93

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Figure D.5 SCDOT Maintenance Cost Schedule 5

SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION
HIGHWAY MAINTENANCE MANAGEMENT SYSTEM
Maintenance Work Description Cost Distribution By StateWide

Organization Unit:99010 - STATEWIDE MAINTENANCE SUMMARY Fiscal Year:JULY10-JUNE11 From Month:JUL 2010 To Month: JUN 2011

	Accomplishment		Cost Distribution				
Activity	Amount	UOM	Labor	Equipment	Material	Total	Unit Cost
PREVENTATIVE MAINT INSPECTION	1,237.000	EACH	29,485	9,135	451	39,071	31.59
REBUILD	57.000	EACH	32,276	8,554	1,507	42,337	742.75
REPAIR	1,254.000	EACH	147,215	32,896	10,870	190,981	152.30
REVISE	191.000	EACH	32,058	11,752	1,155	44,965	235.42
TROUBLE CALL(AFTER HOURS)	117.000	EACH	3,083	778	27	3,888	33.23
606 - PAVEMENT MARKING							
NEW INSTALL/REVISE	43,087,362.750	LF	446,304	180,050	1,599,659	2,226,013	.05
REMOVE	455,414.220	LF	10,831	3,117		13,948	.03
REPLACE	13,772,973.282	LF	195,086	81,964	398,274	675,324	.05
607 - HAND PLACE MARKINGS							
NEW INSTALL/REVISE	3,912.000	EACH	123,707	25,408	54,191	203,306	51.97
REMOVE	1,419.000	EACH	8,978	1,989	9	10,976	7.74
REPLACE	3,547.000	EACH	99,941	24,236	40,976	165,153	46.56
610 - GUARDRAIL							
NEW INSTALL	220,928.090	LF	6,172	1,980	8,025	16,177	.07
REMOVE	3,053.000	LF	32,319	8,493		40,812	13.37
REPAIR	208,542.735	LF	307,575	83,064	153,456	544,095	2.61
611 - WALLS/FENCE							
INSTALL	1,219.900	LF	13,957	3,780	1,398	19,135	15.69
REMOVE	6,176.582	LF	8,977	2,023	167	11,167	1.81
REPAIR	23,593.500	LF	105,923	21,587	11,006	138,516	5.87
613 - IMPACT ATTENUATORS/TERMIN							
INSTALL	7.000	EACH	716	79	550	1,345	192.14

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Figure D.6 SCDOT Maintenance Cost Schedule 6

SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION
HIGHWAY MAINTENANCE MANAGEMENT SYSTEM
Maintenance Work Description Cost Distribution By StateWide

Organization Unit:99010 - STATEWIDE MAINTENANCE SUMMARY Fiscal Year:JULY10-JUNE11 From Month:JUL 2010 To Month: JUN 2011

Activity	Accomplishment		Cost Distribution				
	Amount	UOM	Labor	Equipment	Material	Total	Unit Cost
REPAIR	13.000	EACH	2,350	330	11,060	13,740	1,056.92
614 - HIGHWAY LIGHTING							
INSPECTION	20.000	EACH	613	95		708	35.40
INSTALL	2.000	EACH	1,142	207	700	2,049	1,024.50
REPAIR	304.000	EACH	63,631	12,508	10,529	86,668	285.09
701 - HAZARDOUS CONDITIONS							
PREPARATION	1,635.920	LN MI	124,236	18,981	7,919	151,136	92.39
PREPARATION/STANDBY	9,163.644	LN MI	587,680	87,218	58,432	733,330	80.03
ROADWAY CLEARING	10,540.956	LN MI	164,308	77,014	246,383	487,705	46.27
SPILL RESPONSE	272.122	LN MI	34,115	10,373	2,986	47,474	174.46
WINTER WEATHER OPERATIONS	184,408.893	LN MI	2,868,950	1,196,581	3,828,857	7,894,388	42.81
800 - BRIDGE CONSTRUCTION							
REBUILD EXISTING	1,945.000	SQFT	19,968	10,782	6,612	37,362	19.21
REPLACE	42,438.000	SQFT	1,005,562	480,916	3,017,965	4,504,443	106.14
801 - DECK REPAIR							
CONCRETE REPAIR	4,984.170	SQFT	154,381	31,727	106,489	292,597	58.71
NON-CONCRETE REPAIR	1,389.290	SQFT	18,186	3,216	510,135	531,537	382.60
802 - BRIDGE RAIL REPAIR							
	4,891.050	LF	80,074	14,451	2,670	97,195	19.87
803 - SUPERSTRUCTURE ELEMENT							
BEAMS	217.000	LF	41,704	9,956	1,738	53,398	246.07
FLOOR BEAMS	330.000	LF	766	178	276	1,220	3.70
STRINGERS	126.000	LF	12,684	4,274		16,958	134.59

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Figure D.7 SCDOT Maintenance Cost Schedule 7

SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION
HIGHWAY MAINTENANCE MANAGEMENT SYSTEM

Maintenance Work Description Cost Distribution By StateWide

Organization Unit:99010 - STATEWIDE MAINTENANCE SUMMARY Fiscal Year:JULY10-JUNE11 From Month:JUL 2010 To Month: JUN 2011

	Accomplishment		Cost Distribution				
Activity	Amount	UOM	Labor	Equipment	Material	Total	Unit Cost
805 - BRIDGE EXPANSION JOINTS							
COLD-POURED SEALS	11,264.000	LF	46,156	11,999	1,943	60,098	5.34
COMPRESSION SEALS	181.110	LF	54,215	11,535	8,880	74,630	412.07
806 - BRIDGE BEARING ASSEMBLIES							
REPAIR STEEL BEARING ASSEMBLY	67.000	EACH	9,507	2,068		11,575	172.76
REPLACE ELASTOMERIC BEARING	0.000	EACH	441	225		666	.00
STEEL SADDLE-CONTINUOUS	6.000	EACH	15,441	5,621	304	21,366	3,561.00
STEEL SADDLE-INDIVIDUAL	172.000	EACH	115,577	31,955	48	147,580	858.02
807 - BRIDGE MAINTENANCE							
CLEAN BEARING ASSEMBLIES/CAPS	4,030.800	PHOURS	91,737	21,243	274	113,254	28.10
CLEAN WEEP HOLES	15,196.400	PHOURS	331,821	59,078	7,368	398,267	26.21
DEBRIS REMOVAL	22,186.155	PHOURS	487,385	118,777	1,627	607,789	27.39
ELECTRICAL REPAIR/INSPECTION	4,116.500	PHOURS	123,326	19,449	26,480	169,255	41.12
FENDER REPAIR	2,961.000	PHOURS	79,408	35,677	7,605	122,690	41.44
MECHANICAL REPAIR	4,891.800	PHOURS	113,941	19,687	7,302	140,930	28.81
SCOUR REMEDIATION	5,445.400	PHOURS	128,888	38,785	47,806	215,479	39.57
808 - MOVEABLE SPAN BRIDGES							
	29,421.500	PHOURS	519,470	852		520,322	17.69
OPERATION	12,976.000	PHOURS	236,110	228		236,338	18.21
PREVENTATIVE MAINTENANCE	234.500	PHOURS	7,048	918	1,156	9,122	38.90
REPAIR	441.500	PHOURS	13,595	1,943	1,278	16,816	38.09
809 - BRIDGE PILES AND CAPS							
CAP REPAIR	4.000	EACH	20,554	6,535	3,387	30,476	7,619.00
PILE REPAIR	598.000	EACH	359,938	93,124	45,195	498,257	833.21

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Figure D.8 SCDOT Maintenance Cost Schedule 8

SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION
HIGHWAY MAINTENANCE MANAGEMENT SYSTEM

Maintenance Work Description Cost Distribution By StateWide

Organization Unit:99010 - STATEWIDE MAINTENANCE SUMMARY Fiscal Year:JULY10-JUNE11 From Month:JUL 2010 To Month: JUN 2011

Activity	Accomplishment		Cost Distribution				
	Amount	UOM	Labor	Equipment	Material	Total	Unit Cost
PILE REPLACEMENT	470.000	EACH	575,220	271,454	127,221	973,895	2,072.12
815 - BRIDGE INSPECTION							
BRIDGE INSPECTION	6,593.000	EACH	989,840	93,870		1,083,710	164.37
901 - TRAINING							
ENVIRONMENTAL	1,115.700	PHOURS	30,208	1,686		31,894	28.59
EQUIPMENT RODEO	12.000	PHOURS	269			269	22.42
HAZARDOUS CONDITIONS PREP	14,250.300	PHOURS	314,466	54,392		368,858	25.88
MEDICAL SERVICES	7,931.800	PHOURS	180,454	7,653		188,107	23.72
SAFETY	53,608.300	PHOURS	1,227,977	50,110		1,278,087	23.84
WORKFORCE DEVELOPMENT	66,590.000	PHOURS	1,679,366	93,601		1,772,967	26.63
902 - ENVIRONMENTAL/SAFETY MANA							
	37,721.900	PHOURS	1,166,845	77,035		1,243,880	32.98
903 - BUILDING AND GROUNDS							
ENVIRONMENTAL CLEANUP	7,569.600	PHOURS	165,738	10,321	363	176,422	23.31
INSPECTIONS	7,195.100	PHOURS	168,315	16,757	322	185,394	25.77
MAINTENANCE/CLEANUP	295,505.600	PHOURS	6,492,653	509,366	40,879	7,042,898	23.83
RENOVATIONS	14,689.100	PHOURS	386,124	59,217	7,831	453,172	30.85
ROUTINE CLEANUP	15.000	PHOURS	298			298	19.87
904 - PERMIT MANAGEMENT							
	79,093.250	PHOURS	2,155,832	185,279		2,341,111	29.60
907 - ADMINISTRATION							
ADMINISTRATIVE WORK	470,894.430	PHOURS	16,097,159	657,376		16,754,535	35.58

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Figure D.9 SCDOT Maintenance Cost Schedule 9

SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION
HIGHWAY MAINTENANCE MANAGEMENT SYSTEM
Maintenance Work Description Cost Distribution By StateWide

Organization Unit:99010 - STATEWIDE MAINTENANCE SUMMARY Fiscal Year:JULY10-JUNE11 From Month:JUL 2010 To Month: JUN 2011

	Accomplishment		Cost Distribution				
Activity	Amount	UOM	Labor	Equipment	Material	Total	Unit Cost
908 - INSPECTIONS							
CONTRACT	131,136.234	CL MI	616,003	98,215		714,218	5.45
CULVERT	982.595	CL MI	7,649	987		8,636	8.79
EQUIPMENT	119.960	CL MI	32,193	7,419		39,612	330.21
FACILITY	223.881	CL MI	26,669	3,045		29,714	132.72
GUARDRAIL	50,335.777	CL MI	197,197	29,166		226,363	4.50
NIGHT TIME	61,310.024	CL MI	235,173	26,875		262,048	4.27
ROADWAY/DRAINAGE	178,829.063	CL MI	1,035,711	161,829		1,197,540	6.70
909 - TRAFFIC CONTROL							
	6,067.214	LN MI	616,659	166,070		782,729	129.01
910 - EQUIPMENT MANAGEMENT							
EQUIPMENT INSPECTIONS	8,268.300	PHOURS	176,049	48,881		224,930	27.20
FUEL SERVICE	2,514.800	PHOURS	55,735	7,764		63,499	25.25
TRANSPORTING EQUIPMENT	24,750.000	PHOURS	553,642	260,404		814,046	32.89
920 - STOCKPILE MANAGEMENT							
	39,656.800	PHOURS	882,297	353,244	1,083	1,236,624	31.18
960 - RADIO MAINTENANCE							
BASE INSTALLATION	41.000	EACH	5,540	691		6,231	151.98
BASE REPAIR	353.000	EACH	28,311	3,695		32,006	90.67
BASE-PREVENTIVE MAINTENANCE	292.000	EACH	16,875	2,425		19,300	66.10
BASE-TOWER MAINTENANCE	95.000	EACH	19,929	2,330		22,259	234.31
MOBILE INSTALLATION	325.000	EACH	35,657	4,443		40,100	123.38
MOBILE REPAIR	1,883.000	EACH	96,976	8,532		105,508	56.03
MOBILE-PREVENTIVE MAINTENANCE	1,807.000	EACH	100,764	14,156		114,920	63.60

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Figure D.10 SCDOT Maintenance Cost Schedule 10

SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION
HIGHWAY MAINTENANCE MANAGEMENT SYSTEM

Maintenance Work Description Cost Distribution By StateWide

Organization Unit:99010 - STATEWIDE MAINTENANCE SUMMARY Fiscal Year:JULY10-JUNE11 From Month:JUL 2010 To Month: JUN 2011

Activity	Accomplishment		Cost Distribution				
	Amount	UOM	Labor	Equipment	Material	Total	Unit Cost
OTHER ELECTRONIC REPAIRS	450.000	EACH	28,886	3,731		32,617	72.48
PORTABLE REPAIR	140.000	EACH	5,239	508		5,747	41.05
PORTABLE-PREVENTIVE MAINTENANC	116.000	EACH	3,062	390		3,452	29.76
TOWER INSTALLATION	3.000	EACH	1,314	104		1,418	472.67
TOWER REPAIR	42.000	EACH	4,535	478		5,013	119.36
970 - EQUIPMENT REPAIR							
	232,888.800	PHOURS	6,098,613	159,562		6,258,175	26.87
991 - EQUIPMENT ADMINISTRATIVE							
	183,361.500	PHOURS	5,101,739	71,284		5,173,023	28.21
		Total:	111,350,748	26,723,963	33,648,471	171,723,182	

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Figure D.11 SCDOT Maintenance Cost Schedule 11

Appendix E

GVW1, GVW2 and GVW3 Trucks Bridge Cost per Mile Calculation

The bridge cost per mile for GVW1, GVW2, GVW3 trucks in each axle group was computed as follow:

For GVW1:

$$C_{Pj,1} = \frac{C_{1j}}{VMT_{j,1}} \qquad (E.1)$$

For GVW2:

$$C_{Pj,2} = \frac{C_{2j}}{VMT_{j,2}} \qquad (E.2)$$

For GVW3:

$$C_{Pj,3} = \frac{C_{3j}}{VMT_{j,3}} \qquad (E.3)$$

where,

$C_{Pj,1}, C_{Pj,2}, C_{Pj,3}$: GVW1, GVW2 and GVW3 truck bridge cost per mile in each axle group, respectively

C_{1j}, C_{2j}, C_{3j}: Daily bridge cost allocated to the GVW1, GVW2 and GVW3 trucks in each axle group, respectively

$VMT_{j,1}, VMT_{j,2}, VMT_{j,3}$: Daily VMT (vehicle miles travelled) by the GVW1, GVW2 and GVW3 trucks in the axle group being considered, respectively

j: Axle group

The daily bridge cost allocated to the GVW1, GVW2 and GVW3 trucks in each axle group has two parts: daily fatigue damage cost and daily maintenance cost. The allocation of daily fatigue damage cost was carried out using the fatigue damage contribution of the GVW1, GVW2 and GVW3 trucks in each axle group divided by the total GVW1, GVW2 and GVW3 fatigue damage, respectively.

The daily bridge fatigue damage cost for all bridges in South Carolina is shown in Table E.1. In Table E.1, the annual bridge fatigue damage costs were obtained from Table 9.7.

Table E.1: Daily Bridge Fatigue Damage Cost in South Carolina.

Archetype Bridge	Annual Bridge Fatigue Damage Cost (Dollar)	Daily Bridge Fatigue Damage Cost (Dollar)
A1	3,365,836	9,221
A2	5,554,071	15,217
A3	1,640,698	4,495
A4	627,899	1,720
Others	18,161,514	49,758
All	29,350,017	80,411

The daily bridge fatigue damage for GVW1, GVW2 and GVW3 of each axle group which can be found in Table 9.2 to Table 9.5 for the four Archetype bridges are shown in Table E.2 to Table E.4.

Using the daily cost in Table E.1 multiplied by the respective daily bridge fatigue damage, the daily bridge fatigue damage cost for GVW1, GVW2 and GVW3 of each axle group were calculated and presented in Table E.5 to Table E.7.

Table E.2: Bridge Fatigue Damage Percentage of GVW1 Trucks in Each Axle Group.

Axle Group	A1 GVW1 Damage Percentage	A2 GVW1 Damage Percentage	A3 GVW1 Damage Percentage	A4 GVW1 Damage Percentage	Others GVW1 Damage Percentage
2-Axle	8.607%	1.057%	1.964%	1.519%	3.287%
3-Axle	5.639%	3.686%	3.638%	3.216%	4.045%
4-Axle	4.551%	2.621%	2.940%	2.364%	3.119%
5-Axle	72.937%	38.589%	61.728%	69.394%	60.662%
6-Axle	1.124%	1.105%	0.805%	1.187%	1.055%
7-Axle	1.110%	1.514%	1.091%	1.738%	1.363%
8-Axle	0.010%	0.012%	0.008%	0.012%	0.010%

Table E.3: Bridge Fatigue Damage Percentage of GVW2 Trucks in Each Axle Group.

Axle Group	A1 GVW2 Damage Percentage	A2 GVW2 Damage Percentage	A3 GVW2 Damage Percentage	A4 GVW2 Damage Percentage	Others GVW2 Damage Percentage
2-Axle	0.0009%	0.0004%	0.0005%	0.0006%	0.0006%
3-Axle	0.0037%	0.0080%	0.0069%	0.0048%	0.0059%
4-Axle	0.0005%	0.0007%	0.0008%	0.0007%	0.0007%
5-Axle	3.7171%	6.9076%	7.3094%	9.1257%	6.7649%
6-Axle	0.0528%	0.2867%	0.1516%	0.2518%	0.1857%
7-Axle	0.0666%	0.8515%	0.3771%	0.4873%	0.4456%
8-Axle	0.0170%	0.1845%	0.1314%	0.1142%	0.1118%

Table E.4: Bridge Fatigue Damage Percentage of GVW3 Trucks in Each Axle Group.

Axle Group	A1 GVW3 Damage Percentage	A2 GVW3 Damage Percentage	A3 GVW3 Damage Percentage	A4 GVW3 Damage Percentage	Others GVW3 Damage Percentage
2-Axle	0.0009%	0.0028%	0.0011%	0.0010%	0.0015%
3-Axle	0.0012%	0.0057%	0.0030%	0.0033%	0.0033%
4-Axle	0.0005%	0.0069%	0.0035%	0.0028%	0.0034%
5-Axle	2.1505%	42.8253%	19.4067%	10.2255%	18.6520%
6-Axle	0.0010%	0.0116%	0.0146%	0.0156%	0.0107%
7-Axle	0.0047%	0.2380%	0.3480%	0.2306%	0.2053%
8-Axle	0.0044%	0.0866%	0.0715%	0.1045%	0.0668%

Table E.5: Daily Bridge Fatigue Damage Cost Allocated to GVW1 Trucks in Each Axle Group.

Axle Group	A1 GVW1 Damage Cost (Dollar)	A2 GVW1 Damage Cost (Dollar)	A3 GVW1 Damage Cost (Dollar)	A4 GVW1 Damage Cost (Dollar)	Others GVW1 Damage Cost (Dollar)	Total GVW1 Damage Cost (Dollar)
2-Axle	794	161	88	26	1,635	2,704
3-Axle	520	561	164	55	2,013	3,312
4-Axle	420	399	132	41	1,552	2,543
5-Axle	6,726	5,872	2,775	1,194	30,184	46,750
6-Axle	104	168	36	20	525	854
7-Axle	102	230	49	30	678	1,090
8-Axle	1	2	0.37	0.20	5	8

Table E.6: Daily Bridge Fatigue Damage Cost Allocated to GVW2 Trucks in Each Axle Group.

Axle Group	A1 GVW2 Damage Cost (Dollar)	A2 GVW2 Damage Cost (Dollar)	A3 GVW2 Damage Cost (Dollar)	A4 GVW2 Damage Cost (Dollar)	Others GVW2 Damage Cost (Dollar)	Total GVW2 Damage Cost (Dollar)
2-Axle	0.08	0.06	0.02	0.01	0.30	0.48
3-Axle	0.34	1.22	0.31	0.08	2.91	4.87
4-Axle	0.05	0.11	0.04	0.01	0.35	0.56
5-Axle	342.77	1,051.10	328.56	156.99	3,366.07	5,245.50
6-Axle	4.87	43.62	6.81	4.33	92.42	152.06
7-Axle	6.14	129.57	16.95	8.38	221.72	382.77
8-Axle	1.57	28.08	5.91	1.96	55.62	93.13

Table E.7: Daily Bridge Fatigue Damage Cost Allocated to GVW3 Trucks in Each Axle Group.

Axle Group	A1 GVW3 Damage Cost (Dollar)	A2 GVW3 Damage Cost (Dollar)	A3 GVW3 Damage Cost (Dollar)	A4 GVW3 Damage Cost (Dollar)	Others GVW3 Damage Cost (Dollar)	Total GVW3 Damage Cost (Dollar)
2-Axle	0.09	0.43	0.05	0.02	0.73	1.31
3-Axle	0.11	0.86	0.13	0.06	1.64	2.80
4-Axle	0.05	1.05	0.16	0.05	1.70	3.00
5-Axle	198.31	6,516.57	872.34	175.91	9,280.78	17,043.91
6-Axle	0.09	1.76	0.66	0.27	5.31	8.09
7-Axle	0.43	36.21	15.64	3.97	102.16	158.41
8-Axle	0.41	13.17	3.21	1.80	33.22	51.82

The daily bridge maintenance cost allocated to GVW1, GVW2 and GVW3 of each axle group were calculated in Table E.8 using the percentage of GVW1, GVW2 and GVW3 trucks in each axle group in the total truck population. In Table E.8, the numbers of GVW1, GVW2 and GVW3 trucks in each axle group were calculated using the total ADTT in South Carolina multiplied by their corresponding percentage (see Table 4.2 and Table 4.6). Then the daily bridge maintenance cost was found by using the numbers of GVW1, GVW2 and GVW3 trucks divided by the total ADT in South Carolina and then multiplied them by the daily bridge total maintenance cost in South Carolina (Table E.8).

Table E.8: Daily Bridge Maintenance Cost Allocated to GVW1, GVW2 and GVW3 Trucks in Each Axle Group.

Axle Group	Total ADTT in SC	Total ADT in SC	Daily Bridge Total Maintenance Cost in SC (Dollar)	Number of GVW1 Trucks	Daily Bridge Maintenance for GVW1 Trucks	Number of GVW2 Trucks	Daily Bridge Maintenance for GVW2 Trucks	Number of GVW3 Trucks	Daily Bridge Maintenance for GVW3 Trucks
2-Axle	4,316,773	45,706,454	17,659	381,351	147.34	38	0.01	38	0.01
3-Axle				245,841	94.98	158	0.06	49	0.02
4-Axle				198,429	76.66	21	0.01	20	0.01
5-Axle				3,147,951	1,216.21	158,002	61.04	82,130	31.73
6-Axle				48,089	18.58	2,204	0.85	41	0.02
7-Axle				47,926	18.52	2,752	1.06	171	0.07
8-Axle				427	0.17	702	0.27	166	0.06

The total daily bridge costs for GVW1, GVW2 and GVW3 trucks of each axle group were computed by adding up their corresponding allocated daily bridge fatigue damage cost and allocated daily bridge maintenance cost (Table E.9).

Table E.9: Daily Bridge Cost Allocated to GVW1, GVW2 and GVW3 Trucks in Each Axle Group.

Axle Group	Daily Bridge Cost Allocated to GVW1 Trucks in Each Axle Group (Dollar)	Daily Bridge Cost Allocated to GVW2 Trucks in Each Axle Group (Dollar)	Daily Bridge Cost Allocated to GVW3 Trucks in Each Axle Group (Dollar)
2-Axle	2,851.65	0.49	1.32
3-Axle	3,407.30	4.93	2.82
4-Axle	2,620.10	0.57	3.01
5-Axle	47,966.45	5,306.54	17,075.64
6-Axle	872.17	152.91	8.11
7-Axle	1,108.45	383.83	158.48
8-Axle	8.66	93.40	51.88

Finally, using the daily VMT for GVW1, GVW2 and GVW3 trucks of each axle group shown in Table E.10, the bridge costs per mile for GVW1, GVW2 and GVW3 trucks of each axle group were calculated in Table E.11.

Table E.10: GVW1, GVW2 and GVW3 VMT Distribution in Each Axle Group.

Axle Group	VMT for GVW1	VMT for GVW2	VMT for GVW3
2-Axle	2,474,714	248	248
3-Axle	1,595,344	1,022	319
4-Axle	1,287,670	138	129
5-Axle	20,428,091	1,025,326	532,969
6-Axle	312,068	14,305	264
7-Axle	310,623	17,226	961
8-Axle	2,389	3,925	928

Table E.11: GVW1, GVW2 and GVW3 Trucks Bridge Cost per Mile in Each Axle Group.

Axle Group	GVW1 Trucks Bridge Cost per Mile (Dollar)	GVW2 Trucks Bridge Cost per Mile (Dollar)	GVW3 Trucks Bridge Cost per Mile (Dollar)
2-Axle	0.0012	0.0020	0.0053
3-Axle	0.0021	0.0048	0.0088
4-Axle	0.0020	0.0041	0.0234
5-Axle	0.0023	0.0052	0.0320
6-Axle	0.0028	0.0107	0.0308
7-Axle	0.0036	0.0223	0.1650
8-Axle	0.0036	0.0238	0.0559